Haines Borough

South Portage Cove Harbor Expansion

HARBOR PROTECTION ALTERNATIVES

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Prepared For

Prepared By

PND ENGINEERS, INC.

August 21, 2013

PREFACE TO REPORT

This report has been prepared by PND Engineers Inc., (PND), for the Haines Borough. It presents the results of a study to evaluate the following breakwater alternatives for the South Portage Cove Harbor Expansion project.

i. A rubble-mound breakwater, similar to the existing breakwater, but with wick drains added to improve the foundation so that the soft soils offshore can support the added weight
ii. A steel vertical wall wave barrier, similar to that constructed at that Skagway Small Boat Harbor
iii. A floating breakwater, constructed from concrete caissons, and anchored with chains and/or mooring piles

Included are preliminary design drawings, cost estimates, and the results of advanced numerical models to analyze wave penetration into the harbor, and the strength and consolidation of the soft soils under a rubble-mound breakwater. PND concludes that all three breakwater alternatives are technically feasible, but with different pros and cons and costs. Our recommendation is to proceed with a rubble-mound breakwater if the budget and schedule permits. Rubble-mound breakwaters are a reliable means of protecting a harbor and should have minimal maintenance required if quality rock is used. However, due to the need for foundation improvement at this site and large volume of material needed for construction, a rubble-mound breakwater is the highest cost. A rubble-mound breakwater may also have a longer permitting timeline, and require larger mitigation costs, because of the larger footprint.

A steel pile vertical wall breakwater would be effective and have lower initial cost, shorter construction duration, and a smaller footprint, among other advantages. However, future maintenance costs are likely greater than a rubble-mound breakwater due to costs associated with maintaining the corrosion protection system. There is also a greater risk of wave agitation in the harbor exceeding expectations due to overtopping during storms, and/or wave transmission under the breakwater.

A floating breakwater (wave attenuator) is attractive for many reasons, including the ability to use it for mooring vessels. However, it is also the least effective at blocking waves, its primary purpose. It also has the highest risk of damage from storms or vessel impact, and high maintenance costs. The following table summarizes the alternatives.

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Summary - Harbor Protection Alternatives

	Breakwater Alternative	Initial Cost ($ million)	50 year Life-Cycle Cost	Comments
1	Rubble-Mound (rock) Breakwater w/Wick Drains	$16.0 M	$16.6 M	Wick drains spaced 3 feet on center are needed at an installation cost of approximately $1.5 million to allow the soft sediments to consolidate during a 215 day construction period. The project cannot be feasibly constructed without wick drains. Costs assume relatively expensive locally sourced rock. Costs may be reduced by sourcing rock from outside SE Alaska.
2	Partially Penetrating Wave Barrier	$7.8 M	$8.7 M	The least cost alternative, but with higher maintenance costs than a rubble-mound breakwater because of the need to maintain the corrosion protection system.
3a	Floating Breakwater (Chains/Anchors)	$11.5 M	$12.6 M	A floating breakwater can also provide moorage for vessels, but has the least wave protection, and highest maintenance costs and risk.
3b	Floating Breakwater (Piles)	$11.9 M	$13.3 M	A floating breakwater moored with pile clusters provides better wave protection than a breakwater moored with anchors and chains, but with higher load concentrations and risk.

The recommended next steps for this project are for the Borough to decide on a final design concept for the harbor expansion based on available funding, then to proceed with design, permitting and construction of the preferred breakwater. Thank you for the opportunity to assist the Haines Borough. The report has been prepared by the undersigned.

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ABBREVIATIONS

CLE	Contingency Level Earthquake
FEM	Finite Element Method
FOB	Freight (or Free) On Board
MCE	Maximum Credible Earthquake
NOAA	National Oceanographic and Atmospheric Administration
OLE	Operating Level Earthquake
PND	PND Engineers, Inc.
ROM	Rough-Order-of-Magnitude
USACE	US Army Corps of Engineers

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1. INTRODUCTION

This report presents a study of harbor protection alternatives for the planned South Portage Cove Harbor Expansion (SPCHE) project. PND Engineers, Inc. (PND) has prepared this report for the Haines Borough. Included are concept level design drawings for three types of breakwaters (rock rubble-mound, steel pile vertical wall, and floating). Costs and schedule have been estimated, and the pros and cons analyzed. Our work applies information from previous engineering and design studies for the expansion project, including master plans studies by PND and the US Army Corps of Engineers, and the recently completed geotechnical investigation.

PND recently completed a subsurface geotechnical study that included borings and laboratory soil testing. The results were presented in the report, "Haines Borough, South Portage Cove Harbor Expansion, Draft Final Geotechnical Report", dated April 2013. As part of that study, information was provided for a steel partially penetrating wave barrier and a rubble-mound breakwater. PND recommended finite element modeling (FEM) analysis be conducted to refine estimates of the time to reach limits of acceptable soil movement and stability for the construction of the rubble-mound breakwater option due to the soft soils underlying the breakwater alignment. In addition to evaluating the rubble-mound breakwater and partially penetrating wave barrier options, at the request of the Borough a floating breakwater, as a third option, was included in this study.

Also included in this report is an updated met-ocean study of waves at the project site. Wind data near the harbor was recently made available by the National Oceanographic and Atmospheric Administration (NOAA) which allows for a more refined estimate of wind and waves at the project site.

Attached to this report are appendices presenting detailed analysis and calculations performed for this study. The main body of this report presents the interpretations and conclusions. This report is intended to provide information needed to make a decision on the type of breakwater to be built at South Portage Cove Harbor. The study is not a refined design. Future work is needed to advance the design, including refining the breakwater length and alignment. This study focuses on the following key questions and issues:

- Evaluation of the load vs. deformation behavior of a rubble-mound breakwater and supporting soils during and after construction. PLAXIS FEM software was used to estimate the number and height of stages and the time required to build the rubble-mound with and without wick drains.
- Research and assessment of wick drains, to verify that they can be installed (constructability), the rate of installation, and the potential for damage during installation through sands and gravels.
- Evaluation of breakwater harbor protection, comparing wave conditions in the harbor with and without the breakwater alternatives. This work included new wave computer modeling and wave transmission calculations using Delft3D and CGWAVE.
- Analysis of the anchoring alternatives and mooring loads for a floating breakwater.
- Cost estimates for each harbor protection alternative, including life cycle costs and maintenance needs over an assumed 50 year design life.

1.1 Project Description and Understanding

Portage Cove Harbor is the only full service small boat harbor near Haines, Alaska. There is strong demand for moorage and improved and expanded facilities. Navigation improvements are also needed to accommodate larger commercial-size vessels that are currently constrained by the limited area within the existing breakwater, and the shallow dredged basin depth. The Haines Borough is currently planning a phased approach for improvements to the harbor with the following primary objectives:

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

The most expensive component of the harbor expansion project is extending the breakwater. Portage Cove is exposed to waves generated along long fetch distances in Lynn Canal that can funnel storm-generated waves into the harbor entrance. The existing breakwater is not satisfactory for existing users for some wave conditions, and a longer breakwater is essential if the harbor is to be expanded. Extending the existing rubble-mound breakwater

is one option being considered. Other alternatives are a partially penetrating steel wave barrier, similar to the one recently built in Skagway, and a floating breakwater (wave attenuator).

1.2 Related Studies

As part of this study PND reviewed information from previous projects and studies in the region. The documents include related geotechnical information, wind and wave analysis, and studies by the US Army Corps of Engineers and PND.

Figure 1.1. South Portage Cove – Existing Site

2. GEOTECHNICAL ANALYSIS

The subsurface geology near the tip of the existing breakwater includes a layer of soft clay, as shown in the concept drawings in Appendix A. The layer gets thicker as the water depth increases. This foundation material greatly complicates the design and construction of a rubble-mound breakwater. Alternatives include removing (dredging) the unsuitable material, constructing in phases over a period of years to allow the soft material to consolidate and increase in strength, or improving the foundation by adding stone columns or wick drains to allow the soft sediment to consolidate more quickly as the rock breakwater is constructed. After an initial assessment of alternatives, PND focused on wick drains as a proven technology that would allow a rubble-mound breakwater to be constructed to meet the design criteria for settlement, slope stability and seismic deformation.

2.1 PLAXIS Model of Staged Construction

The rubble-mound breakwater foundation was analyzed using the software program PLAXIS, in addition to desktop calculations of slope stability and consolidation. PLAXIS is a state-of-the-art model typically used for analyzing slope stability, seismic deformation, and settlement. It solves the fundamental soil mechanics and dynamics equations on a grid, using the Finite Element Method (FEM). The model is described in the User's Manual (PLAXIS, 2012). The results are presented in Appendix B.

The PLAXIS analysis calculated the load versus deformation behavior during the construction of a rubble-mound breakwater to determine the number and height of stages and the time required to build the rubble-mound with and without wick drains. A discussion of the analysis methodology and results of the FEM analyses for the rubble-mound breakwater concept are included in Appendix B.

The primary goals of the PLAXIS model were to answer the following questions:

i. Estimate the required staged construction plan, including the number of stages (lifts), height of each stage, and required duration between each stage necessary to maintain an adequate factor against slope failure; and
ii. Estimate the construction time required with and without wick drains.

Based on the FEM analysis, a rubble-mound breakwater without wick drains would require nine stages to construct with the stage heights ranging from about 4 to 8 feet in thickness. Total construction duration of 38 to 81 years would be needed to construct the rubble-mound breakwater in order to meet seismic and slope stability criteria. The incorporation of wick drains will drastically reduce the construction time. Assuming wick drains with 5-foot spacing, the time required for breakwater construction in stages would be approximately 2 years. With 3-foot spacing the construction would be further reduced to about 0.6 years.

2.2 Wick Drains

Wick drains are geotextile filter fabric wrapped around a corrugated plastic core. They are typically about 4 inches wide by 1/4-inch thick. The drains are installed vertically in the ground using a special mandrel mounted on a crane or excavator. The spacing between wick drains is typically between 2.5 and 8 feet for most projects. The drains accelerate the consolidation of soft, compressible soils by providing a pathway for pore water drainage.

A key question is whether wick drain installation is feasible at this project site using conventional technology, considering the layer of dense sands and gravel that they must penetrate. Wick drains are typically installed in soft sediments without having to penetrate dense material. PND investigated wick drain installations means and methods to evaluate constructability. Our assessments included evaluating installation requirements, typical spacing, and time periods required for construction.

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PND initiated discussions with the specialty ground improvement contractor Hayward Baker regarding the feasibility of installing wick drains for this project. Hayward Baker was contacted specifically because of their experience installing wick drains in Alaska and because they are reported to be the only contractor in North America with experience installing wick drains from a barge. Figure 2-1 shows a similar breakwater construction project in Kake, Alaska, where Hayward Baker installed 1,200,000 LF of wick drain from a barge over 25 feet of water to a depth of 90 feet below the ground surface. Figure 2-2 shows a wick drain and mandrel on a PND designed bulkhead project for a Navy base in Umm Qasr, Iraq. The mandrel is driven into the ground with the geotextile wick drain inside, then withdrawn leaving the wick drain in the ground with the end above the ground surface. Figure 2-3 illustrates typical wick drain materials.

Figure 2-1. Installing Wick Drains from a Barge at Kake, Alaska (photo courtesy Hayward Baker).

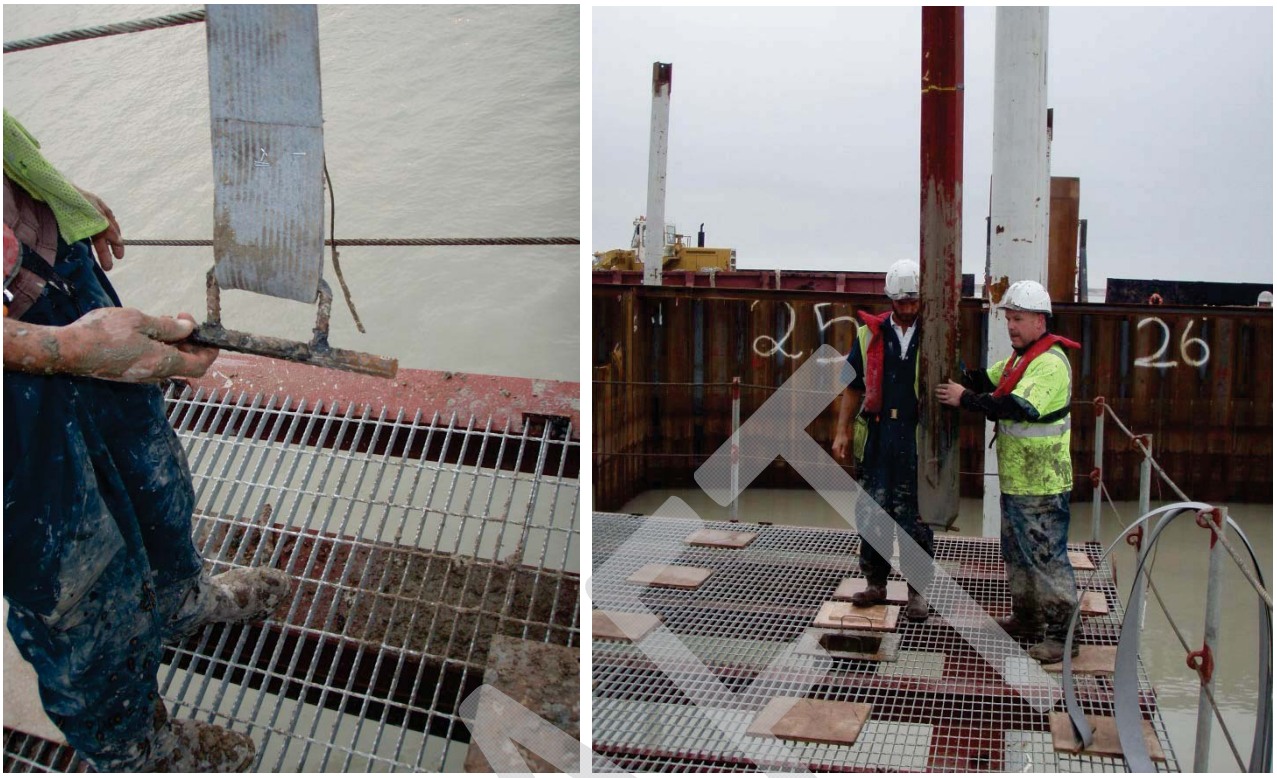

Figure 2-2. Wick Drain and Mandrel – installation from a barge at Umm Qasr, Iraq.

Figure 2-3. Wick Drain Sample

Figure 2-4. Umm Qasr Bulkhead Construction - Sand Layer Placement and Wick Drain Template

The wick drain installation procedure requires a steel mandrel to protect materials from damage. Hayward Baker reviewed the geotechnical information for the site and based on their review and our discussions with them PND is confident that the soil conditions at the site would allow installation of the drains through the upper sand/gravel overlying the clay deposits. PND incorporated wick drains into the rubble-mound breakwater design concept to reduce construction time to a reasonable duration. Alternative foundation improvement methods such as stone columns or dredging/excavation of the clays were not further considered due to their excessive cost relative to wick drains.

Based on our preliminary drain spacing analysis and the PLAXIS numerical model analysis, we estimate that wick drains would reduce the time needed to build the rubble-mound breakwater from multiple years to months. For planning purposes, we anticipate that 3-foot wick drain spacing would result in approximately 215 days total to construct the rubble-mound breakwater, after wick drain installation. Considerations for stage height and duration are discussed in the "Finite Element Model of Rubble-Mound Breakwater Concept" section of this report. A 3-foot triangular grid wick drain spacing, and an average driving distance of 75 feet embedded length, would total about 18,000 drains under the breakwater footprint. The total length would be approximately 1,350,000 feet of installed drains. We estimate that two rigs staged from one barge could complete the wick drain installation in approximately 105 working days at a construction cost of roughly $1,565,000. (This assumes two CAT 375 rigs, shipping FOB to Seattle, including all labor, materials and equipment for installation.)

Additional information and the preliminary analysis of wick drain spacing and performance is included in Appendix E.

2.3 Seismic Deformation Analysis of Rubble-Mound Breakwater Concept

Newmark analysis was used to estimate permanent slope movement under various seismic events (earthquake time histories) for the rubble-mound breakwater alternative. The results of the Newmark analyses show that slope movements on the order of 1/4-inch or less under an Operating Level Earthquake (OLE) event and nearly 4 feet for a Contingency Level Earthquake (CLE) event. No appreciable permanent seismically induced displacements are estimated for the CLE event. Table 2-1 summarizes the result of the seismic deformation analysis. A discussion of the methodology and results are included in Appendix C.

Table 2-1. Summary – Seismic Deformation Analysis

Newmark Displacements	CLE (50% PE in 50 year)	OLE (10% PE in 50 years)	MCE (2% PE in 50 years)
Range (inches)	0	0.01 to 0.25	8.2 to 28.4
Average (inches)	0	0.19	23.9

Note: The average displacement is based on Newmark Rigorous Rigid-Block Analysis.

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3. WIND, WAVE AND WATER LEVELS

The wind, wave and water levels at the project site largely control the size and cost of the harbor protection structures needed. PND analyzed the environmental conditions based on all available information, including previous studies by PND and others. Appendix D presents the design environmental conditions for the project site, as well as analysis of wave transmission past the breakwater and into the planned harbor.

3.1 Met-Ocean Analysis

The met-ocean analysis in Appendix D presents the design environmental conditions for the breakwater alternatives. The met-ocean study for this report has a substantial amount of new analysis, including wave numerical models, and analysis of wind data from the Haines Boat Harbor from 1973 to 1996 which was previously not available from NOAA. The met-ocean analysis in Appendix D provides improved confidence in the estimated design wave height, period, and direction. Figure 3-1 below is an example of wave height output from the CGWAVE numerical model that tested wave penetration into the harbor.

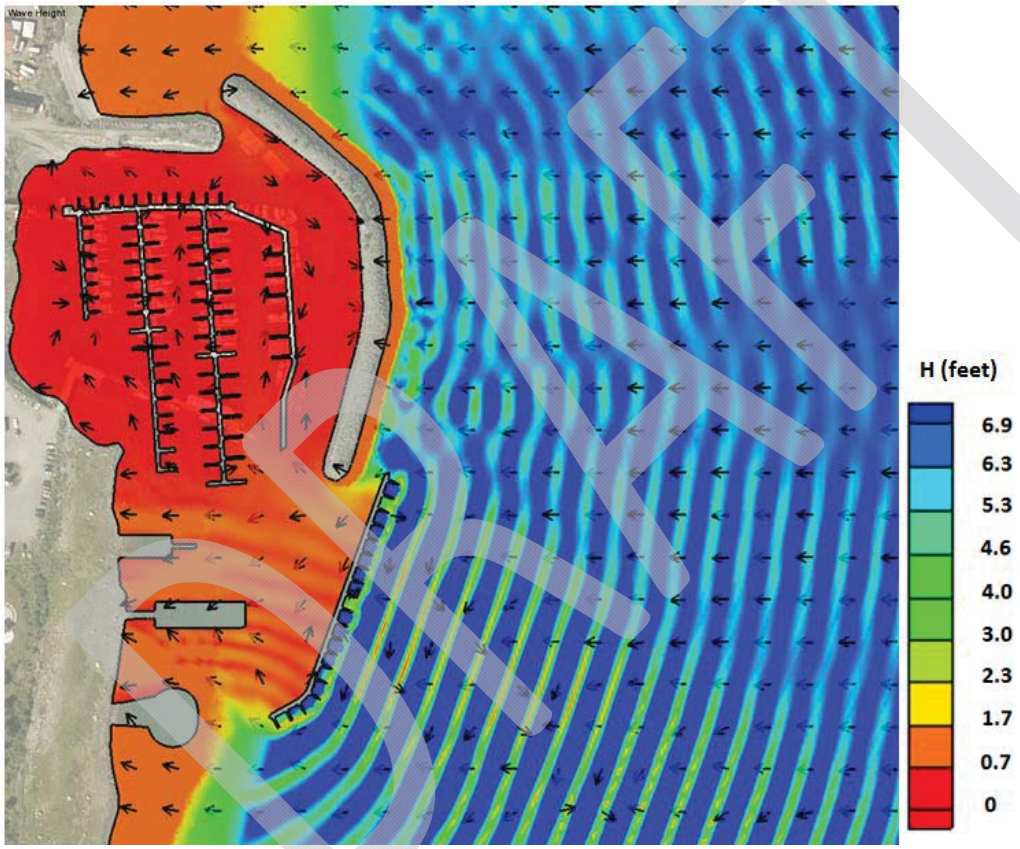

Figure 3-1. CGWAVE Model – Run 8 Sample Output – (Wave Input: H=6.9 feet, T=4.4sec, Dir=090°)

(The wave barrier was modeled as fully reflecting with an assumed transmission coefficient of 0.6).

The recommended Design Operational Conditions (DOC) and Design Environmental Conditions (DEC) for the Portage Cove Marine Facilities are summarized in the tables below. The DEC can be defined as the extreme conditions with a specific combination of tide, wind, waves and currents for which the facilities have to be designed. The DOC is defined as the limiting environmental conditions that would require suspension of normal operations.

Table 3-1. 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 3-2. Portage Cove – Design Environmental 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

The following summarizes the key findings of the met-ocean analysis:

1. Prevailing winds are mostly from the east and northwest. However, large winds also occur from the northeast to southeast. The 50-year return period design wind speed is 68 knots.
2. The 50-year significant wave height is approximately 6.9 feet for winds from the east along a straight line fetch of 3 nautical miles. The waves from the northeast (6.5 feet) and southeast (6.3 feet) are slightly smaller since the waves experience diffraction and refraction effects before reaching the project site.
3. The estimated wind speeds and wave heights are consistent with the previous study performed by USACE in 2004 and local observation. No significant differences were noticed.
4. Tide range, defined as the distance between the Mean Higher High Water and Mean Lower Low Water, is 16.8 feet. The project site is located in an area which is experiencing glacial rebound and relative sea level fall and this should be considered when selecting dredge depths and design water levels.
5. Wave refraction causes large waves from Lynn Canal to approach the breakwater from a mostly easterly direction. Waves penetrate into the harbor through diffraction around the breakwater tip, and transmission under the breakwater (floating and partially penetrating vertical wall). The length and alignment of the breakwater alternatives is reasonable. The rubble-mound breakwater has the most wave protection, and the floating breakwater the least, of the alternatives evaluated.

3.2 Breakwater Wave Transmission

The met-ocean study in Appendix D includes wave transmission analysis and calculations. A rubble-mound breakwater would be more effective than a partially penetrating vertical wall wave barrier, or a floating breakwater at blocking waves from entering the harbor. Predictions of wave transmission under a floating breakwater have the greatest uncertainty, and are largely based on the results of previous physical model studies by others of floating breakwaters.

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4. HARBOR PROTECTION ALTERNATIVES

PND evaluated the following three alternatives for harbor protection:

- Rubble-Mound Breakwater
- Partially Penetrating Wave Barrier
- Floating Breakwater (wave attenuator)

Other breakwaters were considered but dismissed as not feasible or less effective, including a rubble-mound breakwater without wick drains, timber wave fence, and a steel barge floating wave attenuator. The following sections describe each alternative, key issues, and their relative merits.

4.1 Rubble-Mound Breakwater

The rubble-mound breakwater alternative would be similar to the existing rubble-mound breakwater which provides protection to the existing harbor for waves from the north and east. A concept design is shown on the attached drawings in Appendix A. The new rubble-mound breakwater would extend from the end of the existing breakwater nearly 550 feet. The rubble-mound breakwater typical cross-section likely would include the following three layers of rock:

- Core: Shot rock/quarry run material;
- Underlayer: 3-foot-thick layer of larger rock to provide a transition between the overlying armor rock and breakwater core
- Armor Rock: 6-foot-thick layer of the largest rock to protect the structure from wave attack.

In addition, a 3-foot thick layer of sand and/or gravelly sand would be placed on the seafloor prior to constructing the breakwater to serve as a drainage blanket for the wick drains.

Most of the rock placement would likely occur from a barge, either a bottom dump barge and/or placement with a clamshell, crane or excavator. Land-based construction methods might be used via a temporary causeway along the existing breakwater. The contractor's means and methods will influence costs and require further investigation. Environmental permit requirements may also affect the means of rock placement.

To prevent a stability failure of the soft clay layer underlying the site it will be necessary to construct the rubble-mound breakwater in stages. Based on the PLAXIS analysis, the rubble-mound breakwater would likely require nine stages to construct with the stage heights ranging from about 4 to 8 feet in thickness. The duration for constructing the rubble-mound breakwater with wick drains at 5-foot spacing is estimated to be 2 years. Wick drains at 3-foot spacing would require about 0.6 years for rock placement.

During construction of the rubble-mound breakwater the ground settlement must be monitored, typically with survey equipment and settlement plates or other instrumentation. The data is used to determine when consolidation of the underlying clay layer is sufficient to permit placement of additional material without an unreasonable risk of slope failure. The monitoring program should consist of both periodic surveys of the settlement of the mudline elevation under the rubble-mound and the elevation/height of the rubble-mound breakwater construction. PND also recommends that pore-pressure be monitored at various locations beneath the rubble-mound breakwater footprint and within the clay layer. The pressure will increase then decrease after each stage is placed.

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The advantages and disadvantages associated with the rubble-mound alternative are summarized below.

Pros:

- Rubble-mound breakwater construction has been successful at Portage Cove.
- Rock jetties are usually the least expensive type of breakwater if a good quality rock source is nearby and the foundation is suitable.
- Maintenance requirements are likely minimal with the high quality rock available in the region.
- The lack of pile driving is a plus for environmental permitting but may result in a longer work window.

Cons:

- Rubble-mound breakwaters have a larger footprint than other harbor protection alternatives, which, in comparison, can increase the permitting effort and mitigation required.
- The large volume of material required for construction must be delivered to the site via truck or barge, which may cause adverse impacts in town, especially if it coincides with the summer tourist and cruise ship season
- Preliminary cost information from a local quarry indicates a relatively high cost for rock from this source.

4.2 Partially-Penetrating Vertical Wave Barrier

A vertical wall wave barrier can be built using steel pipe piles with flat steel sheet pile wings welded to the pipe to form a combi-wall type of structure. Batter pile clusters (at an angle from vertical) are needed considering the water depths and wave forces at this site. A concept design is shown on the attached drawings in Appendix A. The structure would be similar to the breakwater designed by PND and constructed in 2009 at the Skagway Small Boat Harbor.

Pros:

- It is often the only suitable design when space is limited or the waves are too large for a floating breakwater.
- The small footprint and gap at the bottom for fish passage and circulation likely make this the preferred option for permitting and environmental reasons.
- Construction is relatively fast with minimal truck traffic, noise, and dust.
- The vertical wave barrier at Skagway has been successful and is similar to this concept design.
- A pedestrian promenade can be added to the top at relatively little added cost (not included in cost estimate attached).

Cons:

- More engineering and design effort is needed compared to a rubble-mound breakwater.
- Additional boreholes or probes may be needed to verify the presence of bedrock at the pile tip elevation.
- If bedrock is encountered the piles may need to be socketed in the bedrock.
- Require regular maintenance of anodes and dive inspection to control corrosion.
- Wave overtopping and spray, and/or wave transmission under the breakwater may be large enough to be objectionable. Overtopping can be controlled somewhat by raising the wall height.

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4.3 Floating Breakwater

A concrete float with a rectangular cross-section is typical for the size of structure needed at this site. A concept design is shown on the attached drawings in Appendix A. In deep water, mooring chains are sometimes the only feasible option. Mooring piles are more effective at restraining the floating breakwater and reducing wave transmission than mooring chains. For this structure, PND estimates a pile cluster or mooring chain is needed every 50 feet along the length of the floating breakwater.

Floating breakwaters are most effective for blocking waves with relatively small wave lengths. When wave periods exceed about 3 seconds the cost and size requirements increase greatly. The design wave periods at Portage Cove are close to 5 seconds.

To avoid grounding, the breakwater needs to be placed in deeper water, which increases the size and cost of the mooring system. The concrete float and mooring system are complex and relatively expensive. Considering the wave conditions and tide range at the site, a preliminary assumption is that a floating breakwater would need to be 50 feet wide, 500 feet long, and 10 feet deep to provide effective wave protection at Portage Cove. Additional information is in Appendix D.

Pros:

- A floating breakwater also can be used for mooring small commercial or recreational boats.
- Floating structure moves with the tide and does not block views outside the harbor.
- Can be moved in the future
- Reuse of existing floating structures that may be available (old concrete pontoons, barges, caissons and used breakwaters) is sometimes possible and can be a cost-effective means of providing a breakwater.

Cons:

- The engineering and design is more complex than a rubble-mound breakwater.
- The structure moves and has load concentrations at the mooring hoops and connections, increasing the risk of wear, damage and structural fatigue.
- Floating wave attenuator performance is sometimes difficult to predict accurately without a large scale physical model. The height of the transmitted wave under the float has more uncertainty than a fixed barrier.

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5. COST ESTIMATE

PND estimated the costs for construction and future maintenance cost for each alternative. The estimates are preliminary, with an accuracy of +/- 20%. They are suitable for comparing alternatives based on a Rough Order-Of-Magnitude (ROM) cost estimate.

Table 5-1 below summarizes the cost estimate for each alternative. More detailed cost estimates were developed and are presented in Appendix F. The cost estimates include contingencies and indirect costs such as permitting and engineering and are essentially a recommended project budget. The rubble-mound (rock) breakwater costs are based on recent information provided by a local quarry. Lower cost rock may be available if imported from outside southeast Alaska.

The life cycle cost estimates include regular maintenance and recurring costs over time. The costs are presented in terms of present value dollars to allow direct comparison between each alternative.

Table 5-1. Cost Estimate Summary

	Description	Initial Cost ($ millions)	Maintenance Cost	50 year Life-Cycle Cost
1	Rubble-mound (Rock) with Wick Drains	$16.0 M	$0.6 M	$16.6 M
2	Partially Penetrating Vertical Wave Barrier	$7.8 M	$0.9 M	$8.7 M
3a	Floating Breakwater with Anchors and Chains	$11.5 M	$1.1 M	$12.6 M
3b	Floating Breakwater with Mooring Piles	$11.9 M	$1.4 M	$13.3 M

Note: All costs are in present value dollars

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6. CONCLUSIONS

All three harbor protection alternatives have benefits and drawbacks. Table 6-1 summarizes the three alternatives. In our opinion a rubble-mound structure would be the best alternative if the budget and schedule permits and a good quality rock source is available. All three alternatives are technically feasible, but with different pros and cons.

Table 6-1. Summary - Harbor Protection Alternatives

	Breakwater Alternative	Initial Cost ($ millions)	50 year Life-Cycle Cost	Comments
1	Rubble-Mound (Rock) Breakwater w/Wick Drains	$16.0 M	$16.6 M	Wick drains spaced 3 feet on center are assumed at an installation cost of approximately $1.5 million to allow the soft sediments to consolidate during a 215 day construction period. The project cannot be feasibly constructed without wick drains. Costs assume locally sourced rock. Costs may be reduced by sourcing rock from outside SE Alaska.
2	Partially Penetrating Wave Barrier	$7.8 M	$8.7 M	The least cost alternative, but with higher maintenance costs than a rubble-mound breakwater with high quality rock, because of the need to maintain the corrosion protection system.
3a	Floating Breakwater (Anchors)	$11.5 M	$12.6 M	A floating breakwater can also provide moorage for vessels, but has the least wave protection, and highest maintenance costs and risk.
3b	Floating Breakwater (Piles)	$11.9 M	$13.3 M	A floating breakwater moored with pile clusters has better wave protection than a breakwater moored with anchors and chains, but with higher load concentrations and risk.

6.1 Recommendations for Future Work

This study is based on preliminary engineering and design, at a level suitable for selecting a path forward. During design the following issues should be further investigated to allow for a more refined and efficient design and to minimize risk:

1. Evaluate different bidding and contracting arrangements, such as design-build and the more traditional design-bid-build. A bidding arrangement that allows for more alternatives to be considered could result in a better bid price. It may be advantageous to design and plan for a rock breakwater, but allow a contractor to propose an alternative partially penetrating wave barrier, with the final decision on breakwater type deferred until after the bids/proposals are reviewed.
2. Evaluate sources of rock for the breakwater, including unit costs, shipping costs and available quantities.
3. Review and refine the estimates of optimum wick-spacing vs. construction duration for the rubble-mound breakwater.
4. Investigate likely permit requirements and key constraints. Early discussions with permit agencies can reduce risks and uncertainty and help develop a realistic project schedule.

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5. If a floating breakwater is selected then a physical model in a 3-D wave basin is recommended to test wave transmission and the mooring system for the final design concept.

6.2 Study Limitations

PND's analysis and findings in this report are based the following:

1. Engineering calculations performed by PND;
2. Design criteria developed for this project;
3. Review and application of surveys, geotechnical data and met-ocean analysis; and
4. Engineering judgment and experience, including knowledge gained during design and construction of similar breakwaters and marine facilities in southeast Alaska and other locations.

The information presented in this report is based on professional opinions derived from our analysis and interpretation of available documents and information. This report was prepared by PND for the sole use of the Haines Borough. Our conclusions and recommendations are intended for this project and limitations of scope, schedule and budget apply.

DRAFT

7. REFERENCES

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APPENDIX A

Breakwater Alternatives Figures

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SOUTH PORTAGE COVE HARBOR EXPANSION

BREAKWATER ALTERNATIVES

SITE AERIAL

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TITLE SHEET AND INDEX	1
EXISTING SITE PLAN	2
WIND ROSE & WAVE CRITERIA	3
ALT. 1 - RUBBLE-MOUND BREAKWATER	4
ALT. 2 - VERTICAL WAVE BARRIER	5
ALT. 3A - FLOATING BREAKWATER	6
ALT. 3B - FLOATING BREAKWATER	7

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NOT FOR CONSTRUCTION

REVISIONS

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Juneau, Alaska 99801
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Fax: 907-586-2099
www.pnd-anc.com

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DRAWN: DRH APPROVED: MH
SCALE: AS SHOWN
DATE: August, 2013

HAINES BOROUGH
SOUTH PORTAGE COVE
HARBOR EXPANSION

SHEET TITLE: TITLE SHEET AND INDEX

PN&D PROJECT NO.: 102029.06 DWG. FILE: Title Sheet.dwg

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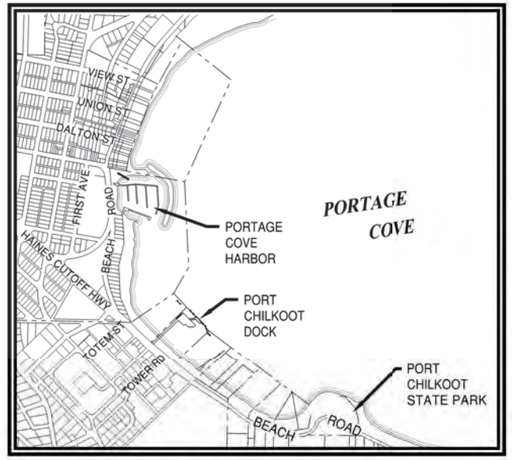
SHEET 1 OF 7



VICINITY MAP

SCALE (FEET)

0 750 1500

LEGEND - GEOTECHNICAL

AP-232 (USACE)	U.S. ARMY CORPS OF ENGINEERS: TEST BOREHOLE LOCATION APPROXIMATE, (2004)
TP 2001 (USACE)	U.S. ARMY CORPS OF ENGINEERS: TEST PIT LOCATION APPROXIMATE, (2000)
TP-8	PND ENGINEERS, INC.: TEST PIT EXCAVATED, (2010)
BH - 2	PND ENGINEERS, INC.: TEST BOREHOLE DRILLED THIS INVESTIGATION, (2012)

NOTES

1. VERTICAL DATUM MLLW = 0.0 FEET.
2. AERIAL PHOTOGRAPHY FROM AEROMETRIC, 2005.
3. BATHYMETRY TAKEN FROM HAINES HARBOR CONDITION SURVEYS PERFORMED IN 2008 AND 2011 BY UNITED STATES ARMY CORP OF ENGINEERS ALASKA DISTRICT (USACE).

EXISTING CONDITIONS

SCALE IN FEET

0 60 120 FT.

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SOUTH PORTAGE COVE HARBOR EXPANSION

SHEET TITLE: SITE PLAN

PND PROJECT NO.: 102029.06 DWG. FILE:

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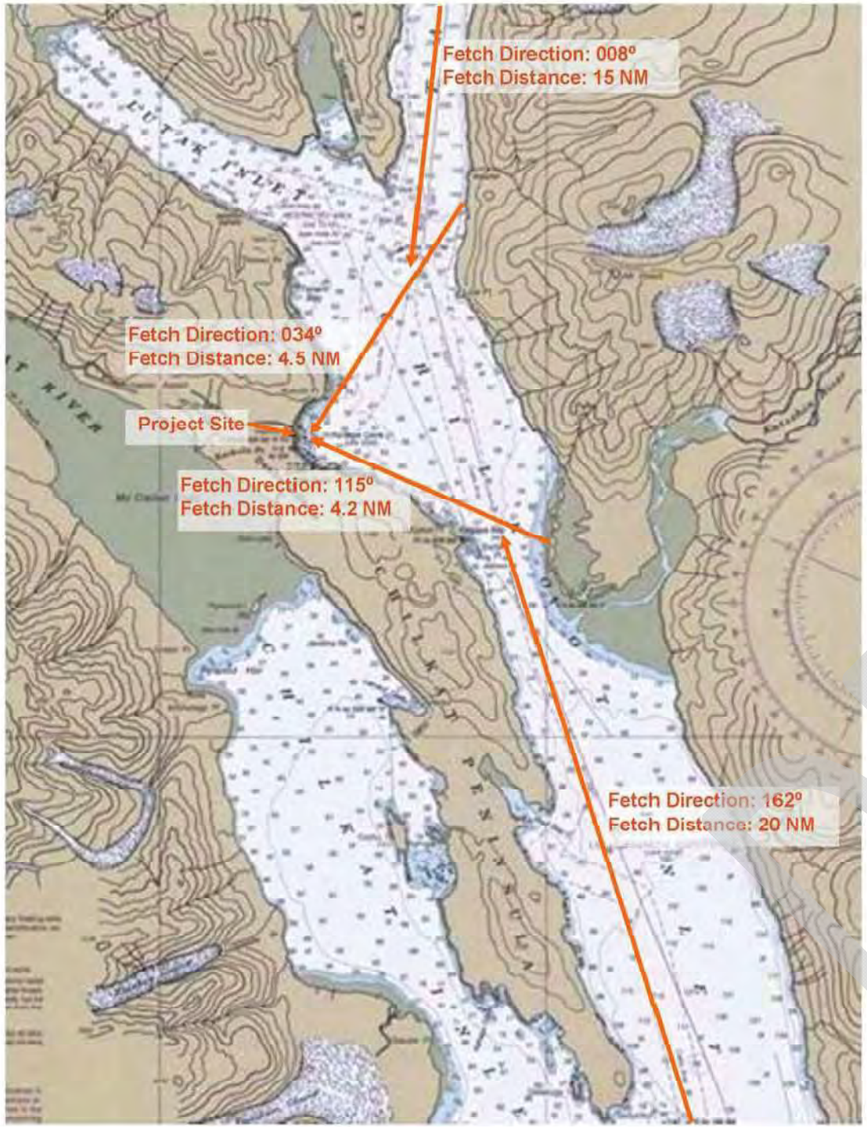
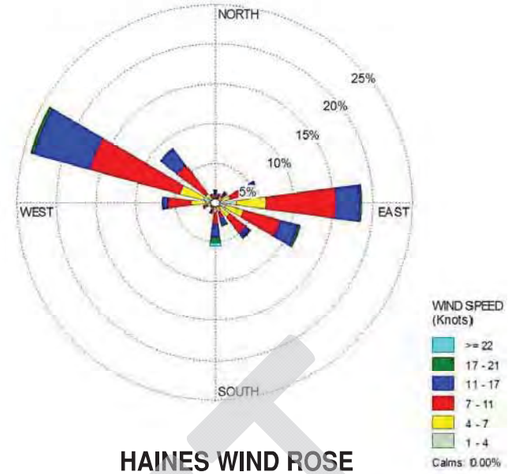
SHEET 2 OF 7



PORTAGE COVE FETCH DIRECTION AND LENGTHS



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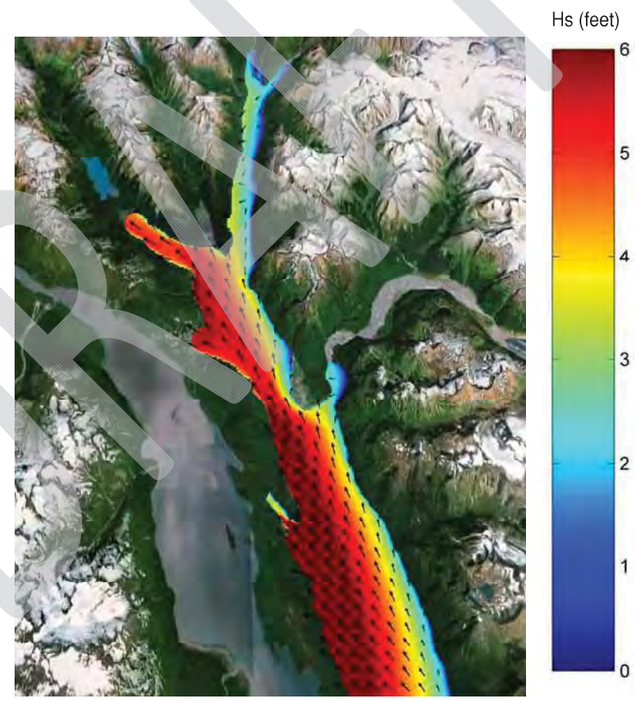
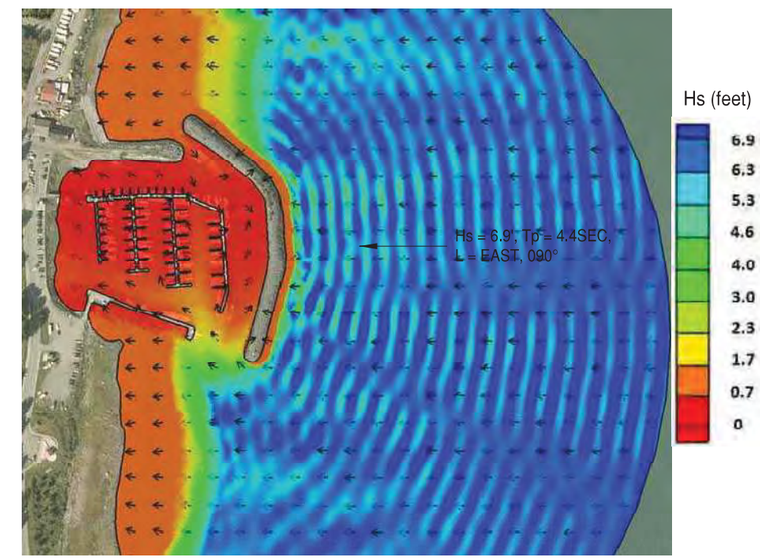


DELFT 3D WAVE NUMERICAL MODEL
50-YR INPUT = 68 KNOT WIND FROM S.E. (120°)



CG WAVE NUMERICAL MODEL
50 YR WAVES FROM EAST (090°)

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SOUTH PORTAGE COVE
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SHEET TITLE: WIND ROSE AND WAVE CRITERIA

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SHEET 3 OF 7

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SOUTH PORTAGE COVE HARBOR EXPANSION

SHEET TITLE: ALTERNATIVE 1 RUBBLE-MOUND BREAKWATER

PND PROJECT NO.: 102029.06 DWG. FILE:

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SHEET 4 OF 7

A SECTION

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HAINES BOROUGH
SOUTH PORTAGE COVE HARBOR EXPANSION

SHEET TITLE: ALTERNATIVE 2 VERTICAL WAVE BARRIER

PND PROJECT NO.: 102029.06 DWG. FILE:

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A SECTION

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HAINES BOROUGH
SOUTH PORTAGE COVE HARBOR EXPANSION

SHEET TITLE: ALTERNATIVE 3A FLOATING BREAKWATER WITH MOORING PILES

PND PROJECT NO.: 102029.06 DWG. FILE:

A

SHEET 6 OF 7

B **SECTION**

NOTE: ANCHOR PILES WITH SPIN FINS MAY OR MAY NOT BE USED AND WILL BE EVALUATED FURTHER DURING DESIGN

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SCALE: SCALE IN FEET 0 80 160 FT.
DATE: August, 2013

HAINES BOROUGH
SOUTH PORTAGE COVE HARBOR EXPANSION

SHEET TITLE: ALTERNATIVE 3B FLOATING BREAKWATER WITH MOORING CHAINS

PND PROJECT NO.: 102029.06 DWG. FILE:

A

SHEET 7 OF 7

APPENDIX B

Finite Element Analysis of Rubble-Mound Breakwater Concept

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APPENDIX B: Finite Element Analysis of Rubble-Mound Breakwater Concept

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B.1 Introduction

This appendix presents the results of the finite element model (FEM) analysis completed for the rubble-mound breakwater concept. The FEM software PLAXIS® was used to model the load versus deformation behavior of the underlying site soils for the construction of rubble-mound breakwater; to determine the number and height of stages and the time required to construct the rubble-mound breakwater. The FEM analysis also evaluated various wick drain configurations beneath the rubble-mound breakwater to increase the rate of consolidation of the underlying soils to reduce the time required to construct the rubble-mound breakwater.

B.2 Analysis Methodology

For the analysis, four generalized subsurface profiles were developed: Sections A-A', B-B', C-C', and D-D', Figures B-2, B-3, B-4, and B-5, respectively. The locations of the profiles are shown on the site plan, Figure B-1. The profiles show the proposed rubble-mound breakwater configuration and the interpreted soil layers underlying the site.

B.2.1 Soil Parameters

Based on the explorations completed at the site, the soils underlying the proposed rubble-mound breakwater alignment consist of three generalized soil layers:

1) Upper Sand – loose, poorly-graded fine- to medium-grain sand with variable silt and gravel content,
2) Clay – very soft to soft, low to medium plasticity clay with localized medium stiff to stiff intervals and intermittent trace angular to sub-round gravel, and
3) Lower Sand/Gravel – dense to very dense with variable silt and gravel content.

Soil strength properties for the site soils used for stability analysis were included in PND's geotechnical report for the project (PND 2013). These soil strength parameters were also used for the FEM analysis and are summarized in the following table, Table B.2-1.

Table B.2-1. Soil Layers and Properties Used in FEM Analysis

Layer Number	Soil Layer	Total Unit Weight pcf	Effective Internal Angle of Friction ϕ' degrees	Effective Cohesion c' psf
1	Upper Sand	125	30	0
2	Clay	120	32	250
3	Lower Sand/Gravel	125	36	0

Figure B-6 presents the undrained shear strength, s_u, profile for the clay layer used in the FEM analysis. The profile was developed using the Stress History and Normalized Soil Engineering Properties (SHANSEP) method (Ladd and Foott 1974) based on the field and laboratory estimates of undrained shear strength and accounts for the stress history of the soil. Based on our interpretation, the top 30 feet of the clay layer is slightly too moderately over-consolidated with an over-consolidation ratio (OCR) ranging from about 1.3 to 7. The equation for the undrained shear strength of the clay layer using the SHANSEP method is:

$$s_u = 0.15 \times OCR^{0.8} \times \sigma'_v$$

Where:

s_u is the undrained shear strength,
OCR is the over-consolidation ratio, and

σ'_v is the vertical effective stress

The coefficients, 0.15 and 0.8, are derived from project specific laboratory and field data and show good agreement with typical values reported in the literature (Ladd 1991, Ladd and DeGroot, 2004).

B.2.2 PLAXIS Model Inputs

PLAXIS 2D® 2012 program is a special purpose two-dimensional FEM program used to perform deformation and stability analysis for various types of geotechnical applications. Models can be either a plane strain or an axisymmetric model depending on the nature of the problem being analyzed. PLAXIS has a number of soil strength models to analyze different soil responses to loading.

B.2.2.1 *Constitutive Soil Models*

The PLAXIS software includes multiple constitutive relationships to model the stress/strain behavior of various soil and rock types. These models have been developed by researchers based on correlated laboratory and field testing and range in complexity and suitability for various geotechnical problems. A review of the various models available in PLAXIS was completed by PND in selecting the appropriate model(s) to represent the behavior of soils underlying the proposed rubble-mound breakwater.

The constitutive relationships reviewed include the Mohr-Coulomb (MC), the Modified Cam Clay (MCC), the Hardening Soil (HS), and the Soft Soil Creep (SSC) models. The simplest constitutive relationship is the MC model which requires minimal inputs to model linear elastic perfectly-plastic stress/strain soil behavior. This model is the most commonly used relationship for predicting soil strength in geotechnical problems and is generally appropriate for 'first-order' approximation of soil and rock behavior. However, the MC model often over predicts deformations due to its simplified stress/strain relationship. Because of the MC model's shortcomings for evaluating undrained loading of soft soils, it was dismissed as an appropriate model to analyze the clay layer underlying the proposed rubble-mound breakwater. In contrast, the MCC and SSC models where developed especially to model primary compression of near normally-consolidated clay soils.

To complete the FEM analysis, the HS model was used to estimate the strength and stability of the soil layers underlying the proposed rubble-mound breakwater. The SSC model was used to model the time dependent deformation behavior of the clay layer from secondary compression. The HS and SSC models require stiffness properties in addition to shear strength parameters of the soil layers to be modeled. Stiffness properties for the upper sand and lower sand/gravel layers were estimated based upon empirical relationships to blow counts (Bowles 1996). For the clay layer, the stiffness parameters were derived from the triaxial and consolidation testing (PND 2013). The stiffness parameters for the clay layer were checked in PLAXIS by using the alternative inputs for the compression index, C_c, and swelling index, C_s, determined from the consolidation test results (PND 2013). The soil parameter inputs into PLAXIS used for the HS and SCC soil models are presented in Table B.2-2 and Table B.2-3, respectively.

Table B.2-2. HS Soil Model Inputs

Layer Number	Soil Layer	Model	E_{50}	E_{oed}	E_{ur}	Poisson's Ratio
1	Upper Sand	Hardening Soil	100,000	115,000	300,000	0.2
2	Clay	Hardening Soil[1]	54,590	43,670	254,300	0.2
3	Lower Sand/Gravel	Hardening Soil	300,000	300,000	900,000	0.3

Note: [1]Model used to evaluate slope stability.

Table B.2-3. SSC Soil Model Inputs

Layer Number	Soil Layer	Model	λ^*	κ^*	μ^*	Poisson's ratio
2	Clay	Soft Soil Creep[1]	0.0478	0.0148	0.0013	0.2

Note: [1]Model used to evaluate time and deformation behavior. See PLAXIS manual for definition of terms.

B.2.2.2 *Drained vs. Undrained Loading and Soil Strength*

The loading of the upper sand and lower sand/gravel layers was modeled as a drained condition; where the rate of loading is assumed to occur at a rate slower than pore water dissipation out of the soil, resulting in no excess pore water pressure generation from an applied load. Therefore, the strength parameters for the upper sand and lower sand/gravel layers input into PLAXIS were the effective, drained, shear strength parameters presented in Table B.2-1.

The loading of the clay layer was modeled as an undrained condition; where excess pore water pressures develop within the clay layer during loading because the rate of loading occurs faster than the pore water dissipation out of the clay layer. Just after loading, when the excess pore water pressures are greatest, the soil is at its lowest strength which corresponds to the undrained shear strength. As the excess pore water pressure dissipate, the shear strength increases as the drained condition is reached. The PLAXIS software provides the framework to estimate soil stresses and excess pore water pressure developed for the given loading over time which is then used to generate the undrained shear strength profile of the soil layer(s). The shear strength versus time is then used to assess the stability of the rubble-mound breakwater during the anticipated construction period.

The PLAXIS software includes three methods for modeling undrained behavior of soils, namely, methods *A, B*, and *C*. In Method *A* the effective shear strength parameters are input and the model generates the undrained shear strength used for the analysis based on the constitutive relationship used to model the soil layer. In Method *B*, the shear strength parameters are input by the user as undrained strengths directly and must be updated with each load increment to account for changes to the undrained shear strength (due to the reduction in the soil void ratio from consolidation from the previous loading increment). Method *C* is a total stress approach with the shear strength input similar to Method *B* but excess pore pressures are not accounted for during the loading increment.

Method *B* was selected for the modeling of the undrained response of the clay layer with the resulting excess pore water pressure and resulting shear strength both reviewed and updated with each load increment. Method *B* allows the user to control the undrained shear strength of the soil rather than the software calculating the excess pore water pressure and computing the strength and resulting factor of safety against failure. Method *B* was used for the analysis to prevent unsafe estimates in staged construction performance using the Method A approach that have been documented by Ladd (1991) and others.

B.2.2.3 *Flow Parameters*

Flow parameters were determined based on published data on soil types (including those found within PLAXIS) and results from the consolidation testing performed on the samples obtained from the boreholes completed at the project site (PND 2013). The vertical permeability of the clay layer input into PLAXIS was calibrated using the consolidation testing results and was estimated to be 0.00094 ft/day. This value is in agreement with published values for similar soil types (Holtz and Kovacs 1981; Cornforth 2005). The horizontal permeability was assumed to be twice that of the vertical permeability to account for natural soils typically having greater permeability in the horizontal direction than in the vertical direction. The change in permeability during loading was based on the compression index for the SSC model as recommended in the PLAXIS manual.

B.2.3 Boundary Conditions

The boundary conditions for the FEM consisted of horizontal fixities at the far left and far right sides of the model to allow translation in the vertical, but not horizontal direction. The model was made wide enough so that the fixed

boundary conditions did not influence the deformations of the rubble-mound breakwater construction and underlying soil layers. The bottom of the model was fixed in both the vertical and horizontal direction to simulate very dense soil and/or bedrock but was placed deep enough to have no significant influence on the deformation calculated by the model. The ground surface and soil layers as well as the rubble-mound breakwater itself were allowed to move freely in both the horizontal and vertical directions.

B.3 Slope Stability

Previously completed limit equilibrium slope stability analysis (PND 2013) indicate that the proposed rubble-mound breakwater would need to be constructed in stages to maintain and adequately safe factor of safety against slope failure due the low undrained shear strength of the clay layer coupled with the relatively large load demand from the rubble-mound breakwater. To further analyze the required stage heights and time required for each stage, two FEMs for each subsurface profile were analyzed using the PLAXIS software. The first model determined the maximum stage height to maintain the factor of safety for slope stability using the undrained shear strength of the clay layer. The second model evaluated the time and deformation behavior of the rubble-mound breakwater and underlying soils resulting from the stage height determined from the first model. These models were iteratively evaluated for each stage of the proposed rubble-mound breakwater construction where the shear strength of the clay layer was updated for each stage as discussed in the Analysis Methodology section of this appendix. A discussion and results of each of these analyses is presented in the following sections.

B.3.1 Stage Construction Heights

The four subsurface profiles, Figures B2 to B5, were analyzed for global stability and it was found that the location and geometry of the failure surfaces and maximum stage heights required to maintain a safe slope condition did not vary significantly amongst the four profiles. For this reason, the analysis was simplified to one generalized subsurface profile, Section C-C' on Figure B4, which is considered the critical section where the clay layer is thickest.

Nine stages were determined to be required for rubble-mound breakwater construction assuming that the entire rubble-mound breakwater would be constructed to the same elevation with each stage. An iterative process was used to determine the maximum height of each stage of construction to satisfy the temporary factor of safety for global stability of 1.3. The iterative process for each stage consisted of developing an undrained shear strength profile of the clay layer based stress history using the SHANSEP procedure and then "building" the embankment in the PLAXIS model to satisfy the temporary construction factor for slope stability. The undrained shear strength profile was updated for each stage to reflect the change in effective vertical stress from the previous stage. This process was then repeated for each stage until the crest elevation of the rubble-mound breakwater was reached. The maximum stage height elevations to maintain the temporary factor of safety against slope failure are summarized in Table B.3-1.

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Table B.3-1. Rubble Mound Stage Construction Elevations

Stage	Elevation, MLLW (feet)
1	-20
2	-12
3	-4
4	2
5	8
6	12
7	16
8	20
9	24

The critical failure surfaces from the slope stability analysis completed in PLAXIS were reviewed and compared to limit equilibrium analysis completed earlier (PND 2013) and the calculated safety factors and failure geometries showed good agreement. Results of the failure surfaces for each stage generated by PLAXIS are presented as Figures B-7 to B-16. It should be noted that the phi-C reduction method for computing the factor of safety produces displacements that have no physical meaning (PLAXIS 2013) however the incremental displacement is used to display the relative displacements in the failure surfaces. Deformation analysis is discussed below.

B.3.2 Time versus Deformation

The analysis of the time versus deformation (settlement) was evaluated for rubble-mound breakwater construction without and with wick drains. These evaluations include estimates for secondary compression over the 50-year design life of the proposed rubble-mound breakwater. All the deformation results presented herein are estimated beneath the centerline of the rubble-mound breakwater (conservative values).

B.3.2.1 *Without Wick Drains*

The resulting time versus deformation during construction for subsurface profiles A-A', C-C', and D-D' are presented in Figure B.3-1 for rubble-mound breakwater construction without wick drains.

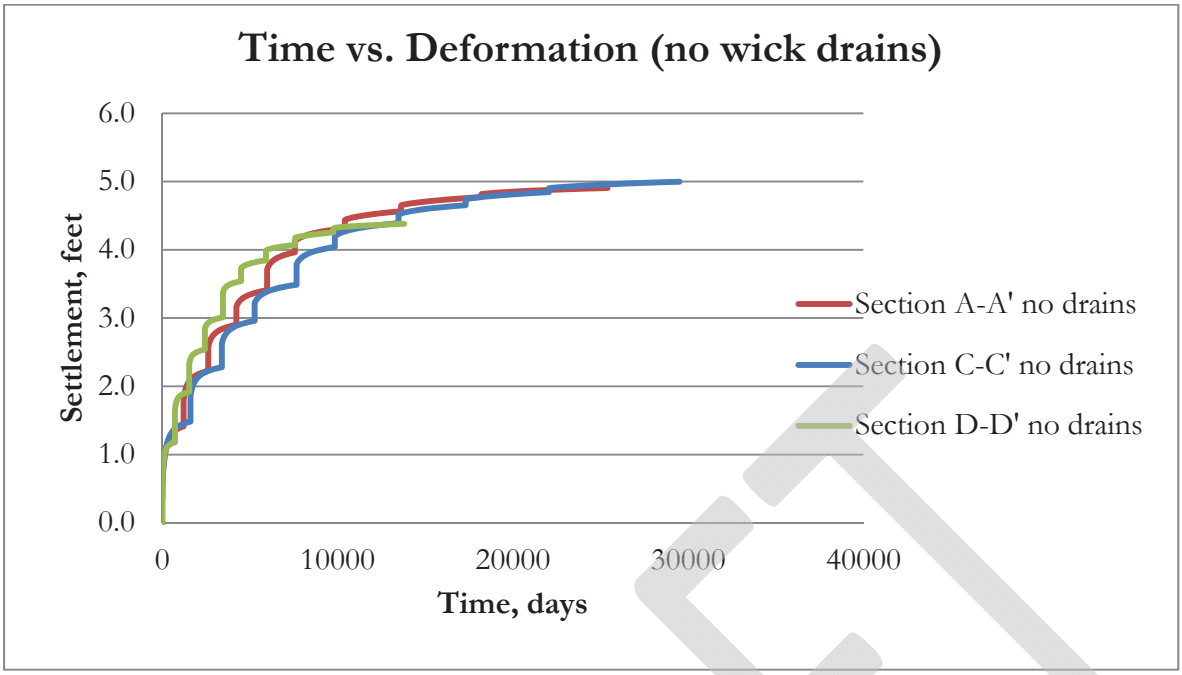

Figure B.3-1. Time vs. Deformation along the Centerline of the Rubble Mound with No Wick Drains

The results of the deformation analysis estimate that the total settlement expected during construction of the proposed rubble-mound breakwater to be on the order of 4 to 5 feet with approximately 3 feet of primary consolidation and another 1 to 2 feet of secondary compression. The time required for each stage was determined when 95% of the primary consolidation had occurred which corresponds roughly to the amount of consolidation required to maintain the minimum factor of safety of 1.3 for slope stability during construction as discussed above. In other words, to maintain a safe slope condition 95% of the excess pore water pressure generated from the previous stage must dissipate before the next stage can be constructed. The time rate of settlement shown in Figure B.3-1 indicates that the time required to achieve 95% consolidation for each stage for construction without wick drains results in a total duration ranging from about 38 to 81 years to build the entire rubble-mound breakwater.

B.3.2.2 *With Wick Drains*

Various wick drain spacing were modeled to determine the benefit to reduce the overall time that would be required to construct the proposed rubble-mound breakwater using the same stage heights determined earlier. The time required for rubble-mound breakwater construction with wick drains at 5 foot spacing is estimated to be nearly 2 years and wick drains at 3 foot spacing would require about 0.6 years. The results are shown in Figure B.3-2 below.

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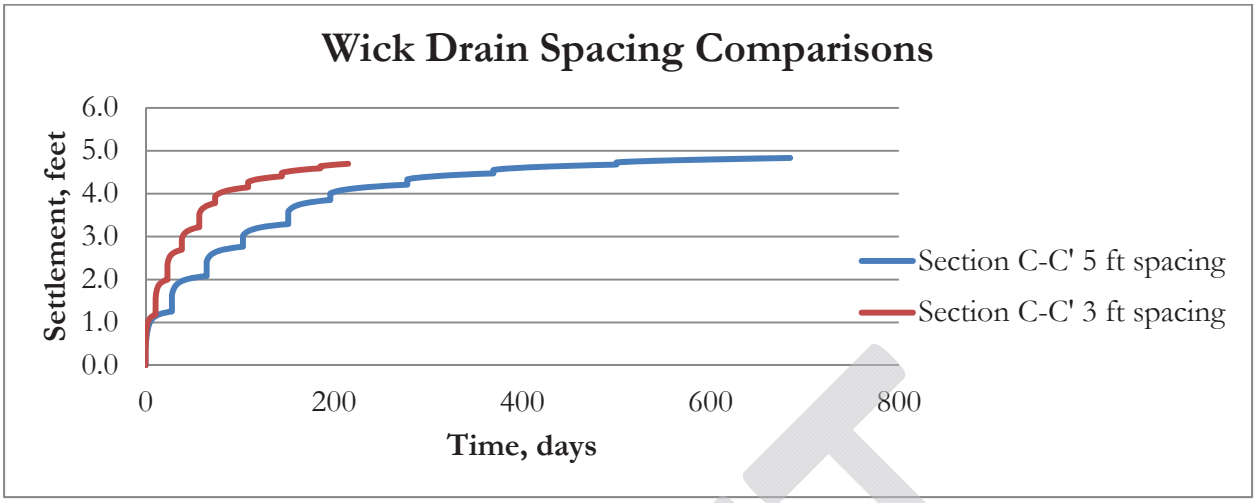

Figure B.3-2. Time vs. Settlement Centerline of Rubble Mound for Wick Drain Spacing Comparison

Section C- C' was considered for comparing the wick drain spacing as the drainage path is simply the distance between the wick drains and no longer a function of the thickness of the clay.

The Time versus Deformation during construction for subsurface profiles A-A', C-C', and D-D' are presented in Figure B.3-3 for rubble-mound breakwater construction with wick drains spaced at 3 feet.

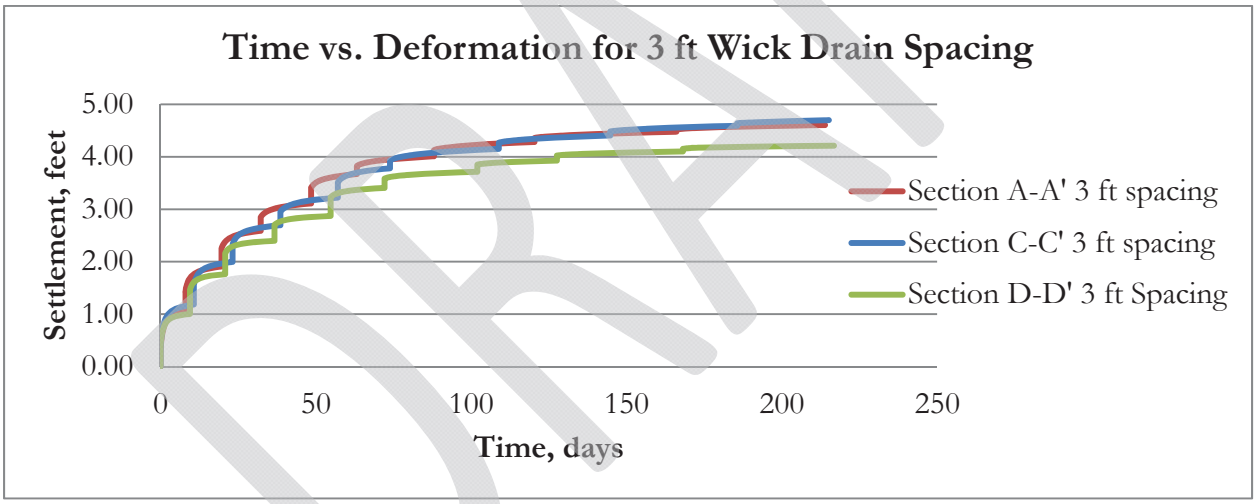

Figure B.3-3. Time vs. Deformation at Centerline of Rubble Mound for 3-Foot Wick Drain Spacing

For 3 foot drain spacing, the time required for construction is approximately 215 days to reach 95% primary consolidation of the last stage.

B.3.2.3 *Long-Term Settlement*

The long term settlement of the rubble-mound breakwater due to secondary compression was estimated using the SCC model in PLAXIS for the 50 years following construction (95% primary consolidation of the last stage) for subsurface profile Sections A-A', C-C', and D-D'. The results are presented in Figure B.3-3.

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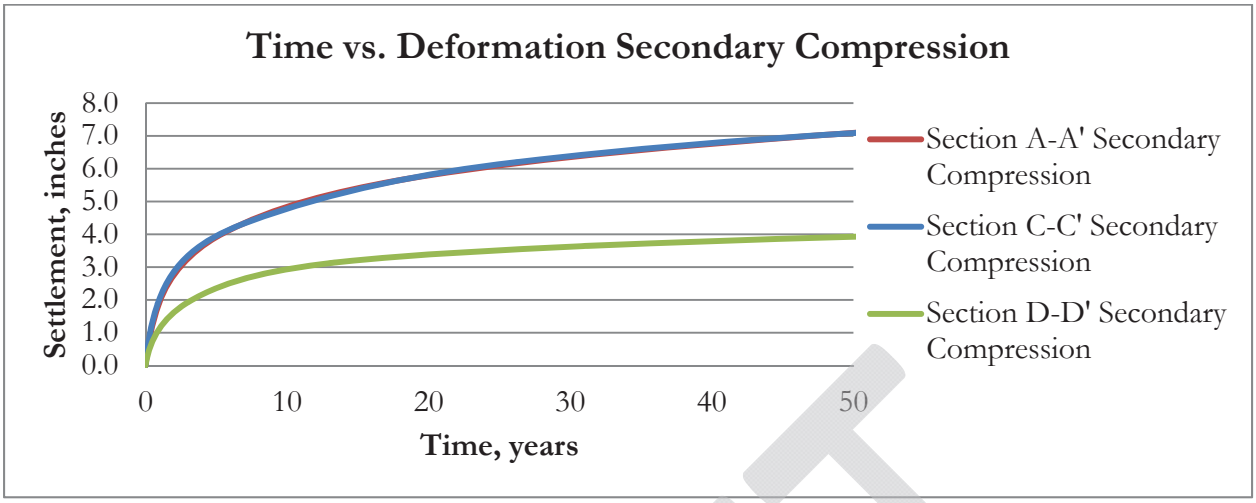

Figure B.3-3. Time vs. Secondary Compression at the Centerline of Rubble Mound

Long-term settlement of the rubble-mound breakwater after 50 years of 4 to 7 inches is estimated depending on the clay layer thickness (Sections A-A' and C-C' had similar clay layer thickness whereas Section D-D' is approixmately half the thickness of Sections A-A' and C-C').

B.3.2.4 *Glacial Rebound*

Glacial rebound is anticipated to occur at a rate of 0.9 inches/year (Greymueller et al. 2008). Over the 50 year design life this is anticipated to result in roughly 45 inches of uplift of the rubble mound breakwater. The installation of wick drains will result in the majority of primary consolidation occuring during the intitial 100 days of rubble mound breakwater construction with almost all of primary consolidation occuring within the 1-year construction timeframe. Figure B.4-3 indicates an additional 4 to 7 inches of secondary compression may occur over the 50 years. The combination of uplift and limited future settlement will not require the crest elevation of the breakwater to be constructed higher to account for settlement. Rebound will actually have an positive effect with respect to the rubble mound breakwater protection. However, consideration may need to be given to dredging depths to account for rebound and other infrastructure for the harbor expansion.

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B.4 References

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Figures – Plan View and Rubble-Mound Breakwater Profiles

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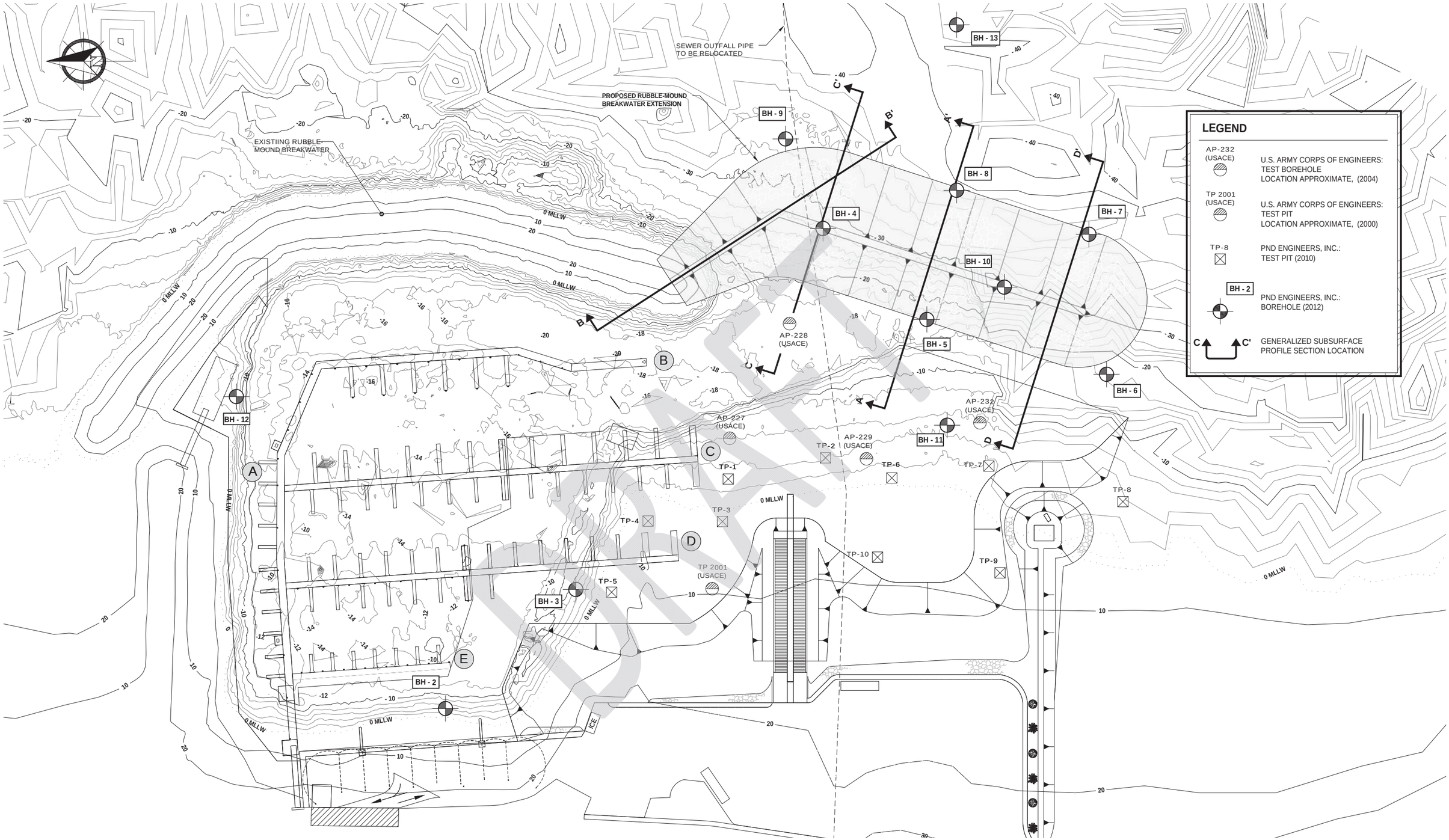

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DATE: August 2013

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SOUTH PORTAGE COVE HARBOR EXPANSION

SHEET TITLE: SITE PLAN WITH PROPOSED RUBBLE-MOUND BREAKWATER

PND PROJECT NO.: 102029.04 DWG. FILE: B Fig1.DWG

B

FIGURE 1

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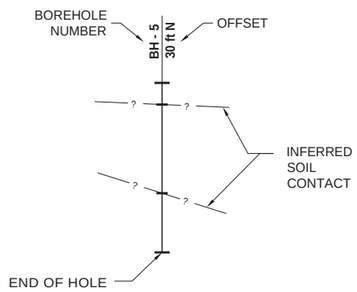

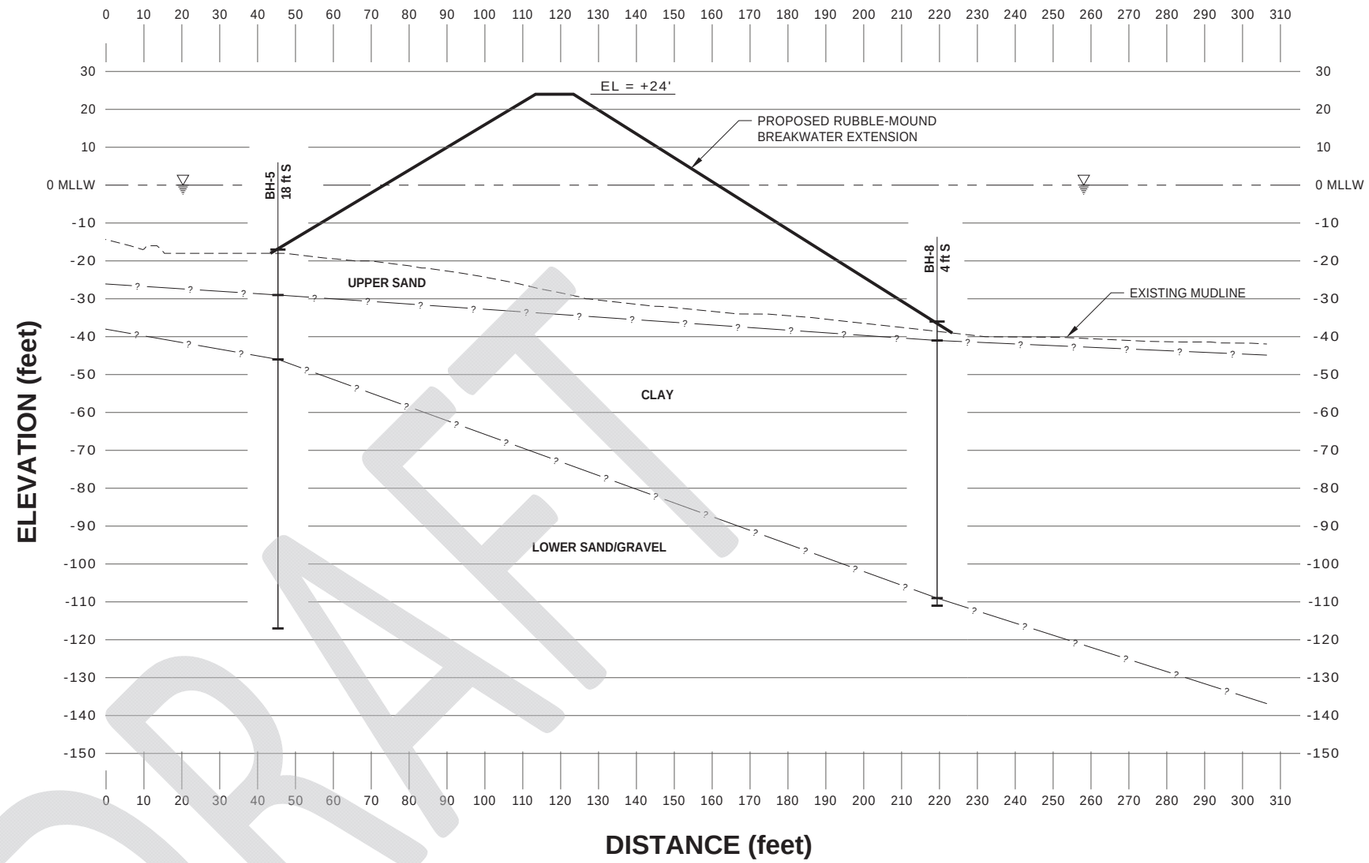

SECTION A-A'

SCALE IN FEET

0 20 40 FT.

NOTE: HORIZONTAL SCALE EQUALS VERTICAL SCALE

NOTES

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3. VARIATION IN MUDLINE ELEVATIONS WERE OBSERVED BETWEEN BATHYMETRIC SURVEY DATA AND LEADLINE MEASUREMENTS TAKEN AT BOREHOLE COLLARS.
4. SEE BOREHOLE LOGS AND GEOTECHNICAL REPORT FOR DETAILED SOIL DESCRIPTIONS.

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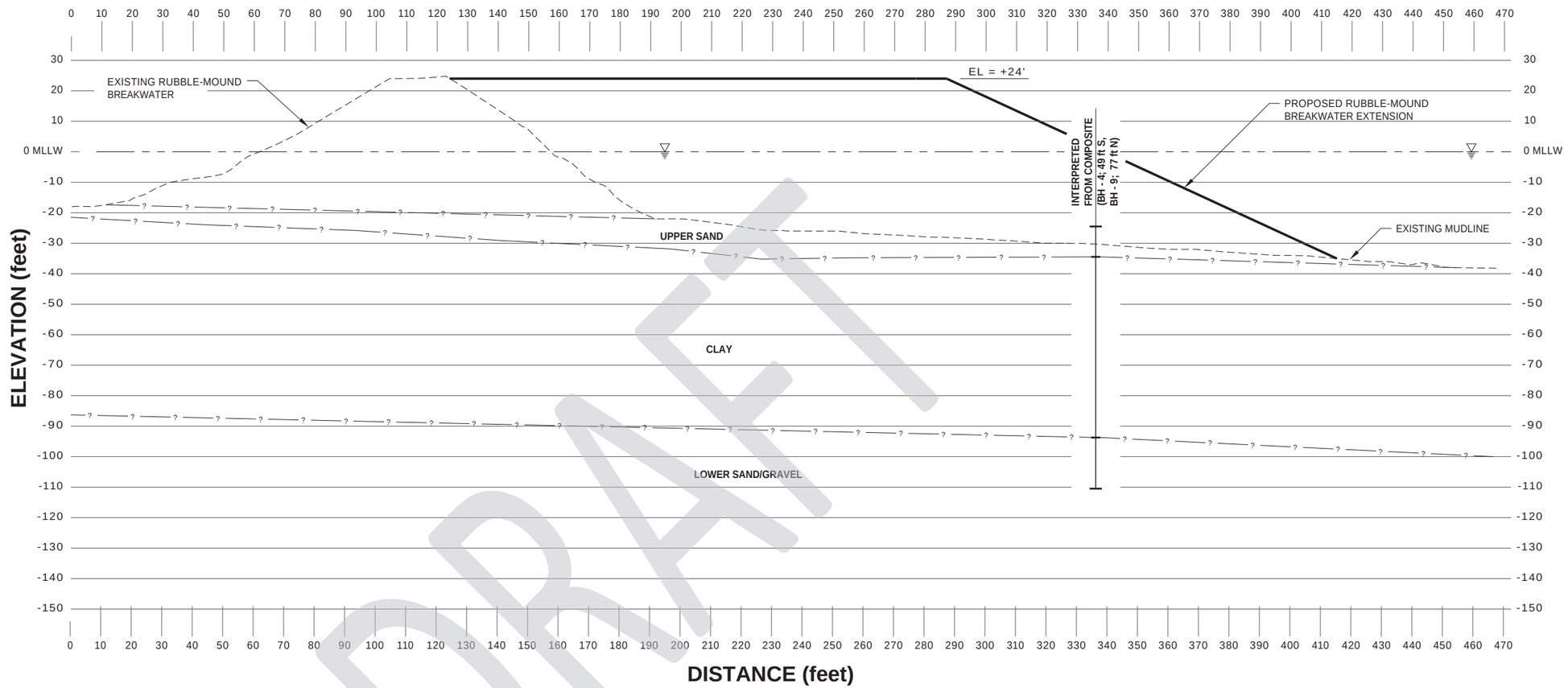
HARBOR EXPANSION

SHEET TITLE: GENERALIZED SUBSURFACE PROFILE SECTION A-A'

PND PROJECT NO.: 102029.04 DWG. FILE: B Fig2.DWG

B

FIGURE 2



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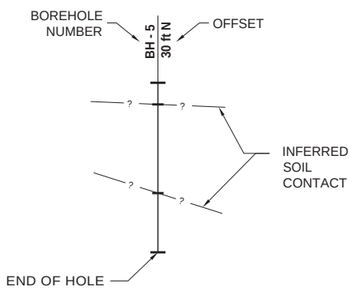

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SECTION B-B'

SCALE IN FEET

0 20 40 FT.

NOTE: HORIZONTAL SCALE EQUALS VERTICAL SCALE

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HARBOR EXPANSION

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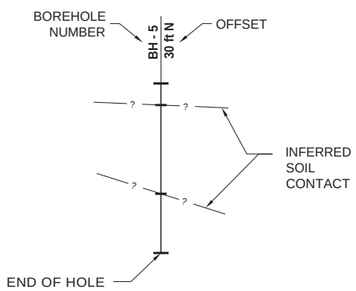
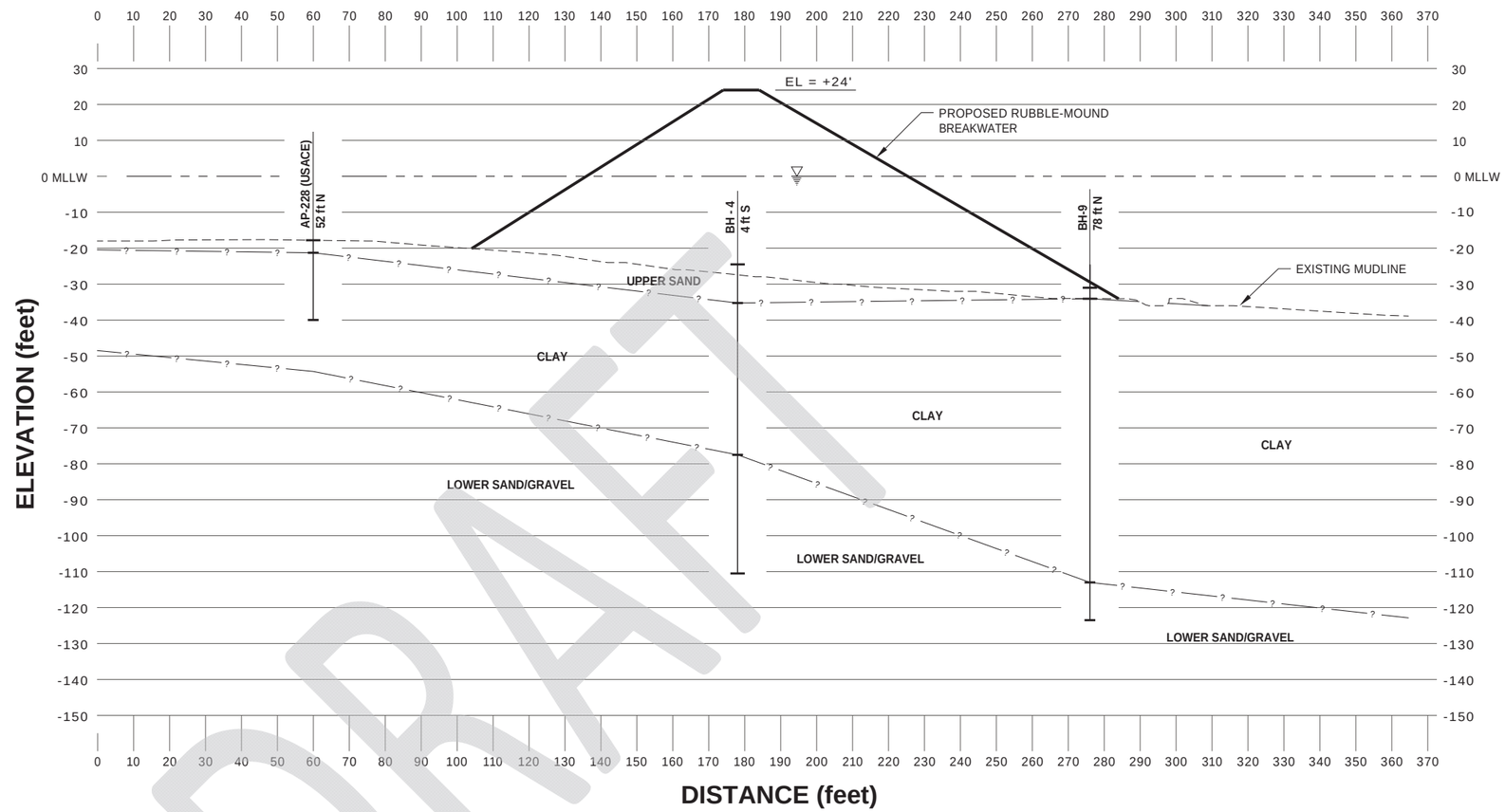
SECTION B-B'

PND PROJECT NO.: 102029.04 DWG. FILE: B Fig3.DWG

B
FIGURE
3

LEGEND





SECTION C-C'

SCALE IN FEET

0 20 40 FT.

NOTE: HORIZONTAL SCALE EQUALS VERTICAL SCALE

NOTES

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HARBOR EXPANSION

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GENERALIZED SUBSURFACE PROFILE
SECTION C-C'

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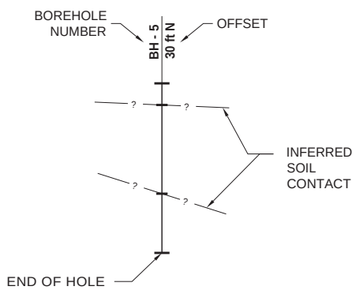
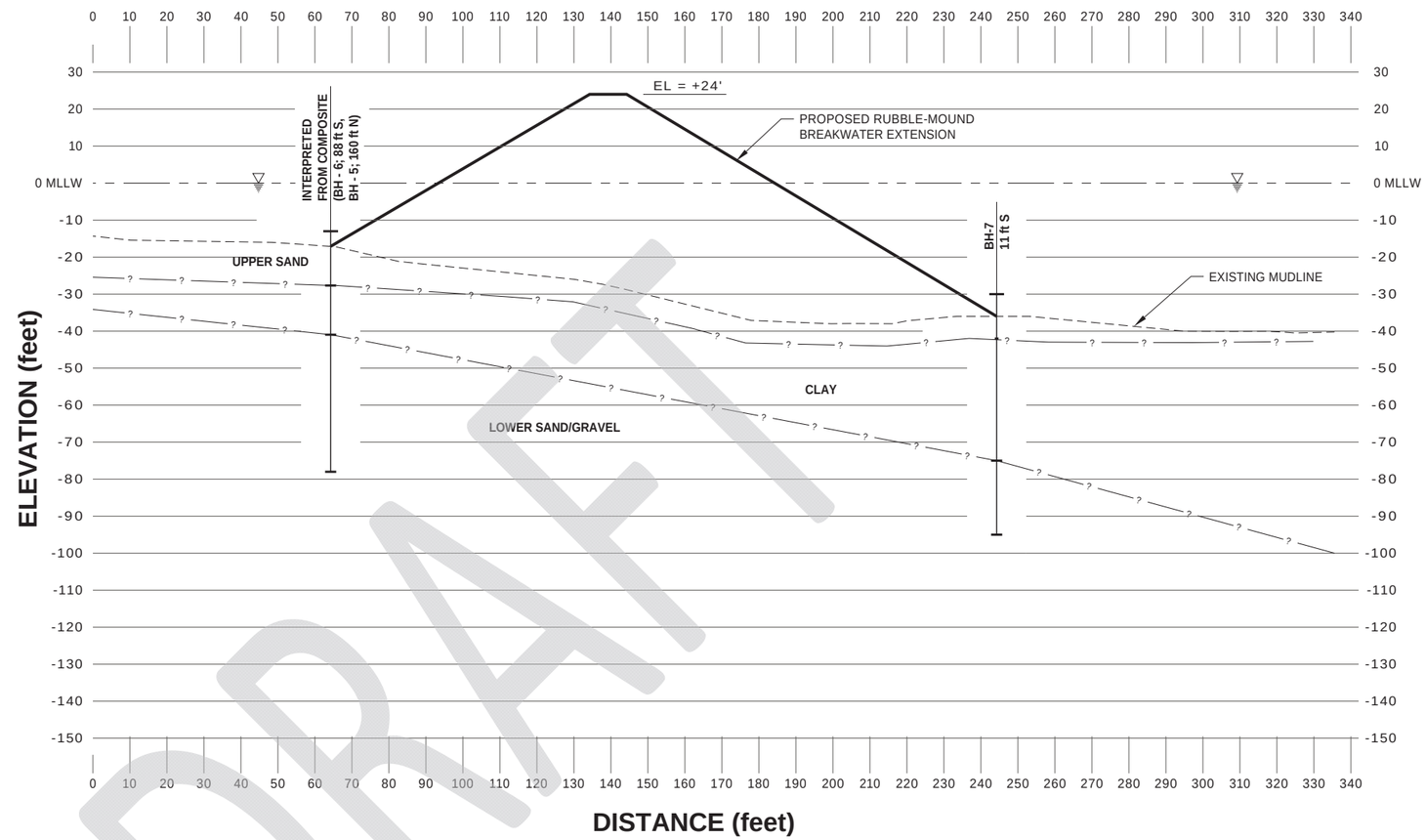
B
FIGURE
4

LEGEND





SECTION D-D'

SCALE IN FEET

0 20 40 FT.

NOTE: HORIZONTAL SCALE EQUALS VERTICAL SCALE

NOTES

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SHEET TITLE:
GENERALIZED SUBSURFACE PROFILE
SECTION D-D'

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B
FIGURE 5

Figures – Undrained Shear Strength Profile

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Undrained Shear Strength Profile

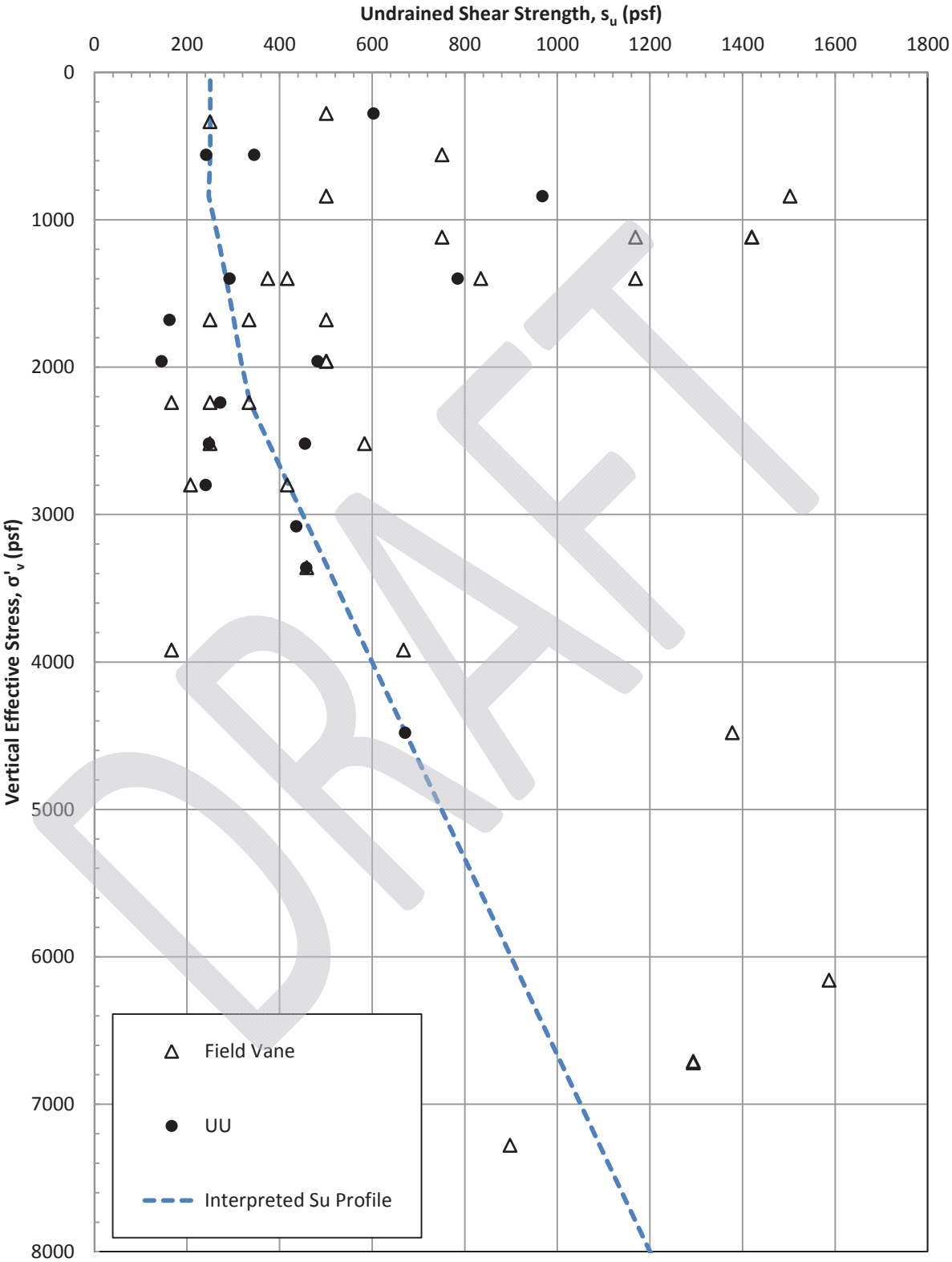

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FIGURE B-6

Figures – PLAXIS Slope Stability Outputs

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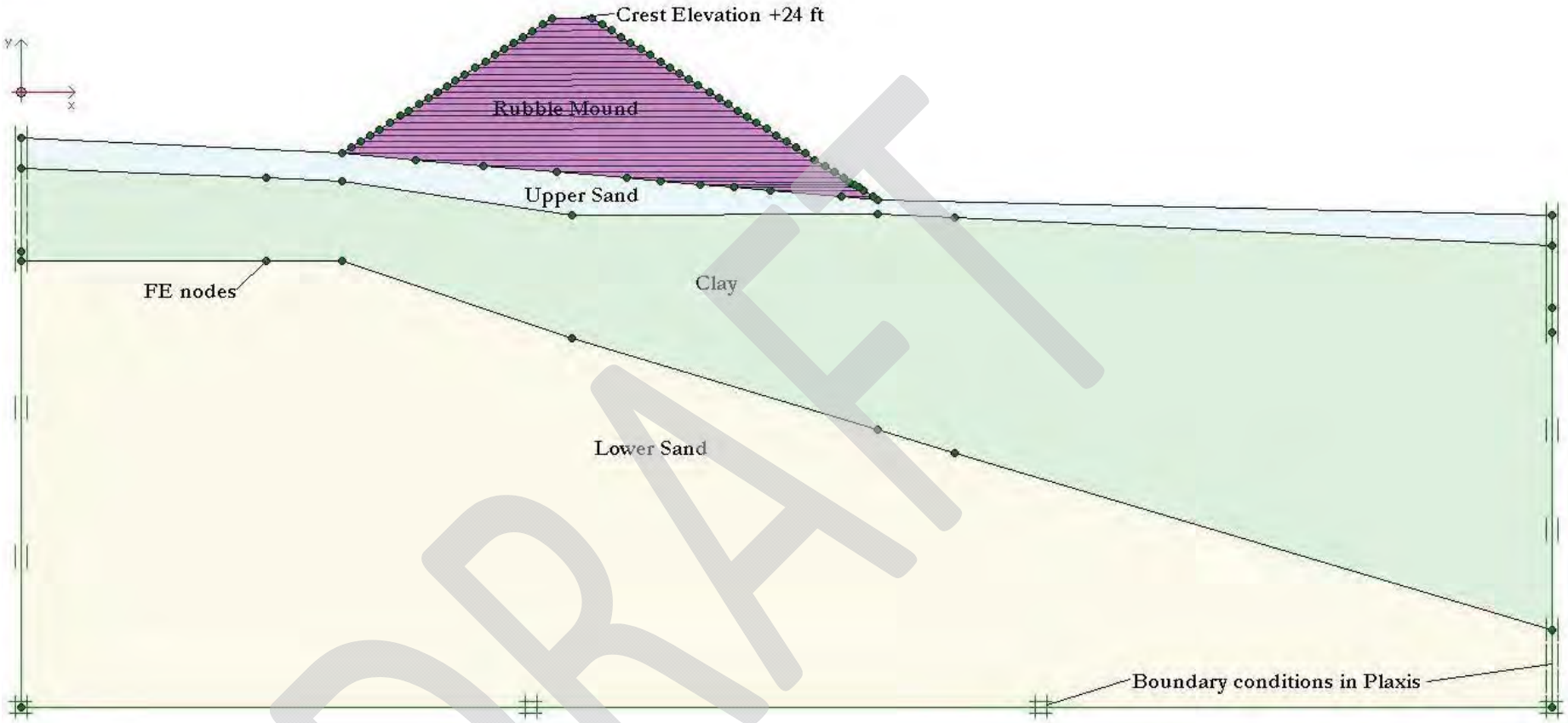

General Rubble-Mound Breakwater Geometry in PLAXIS for Section C-C'

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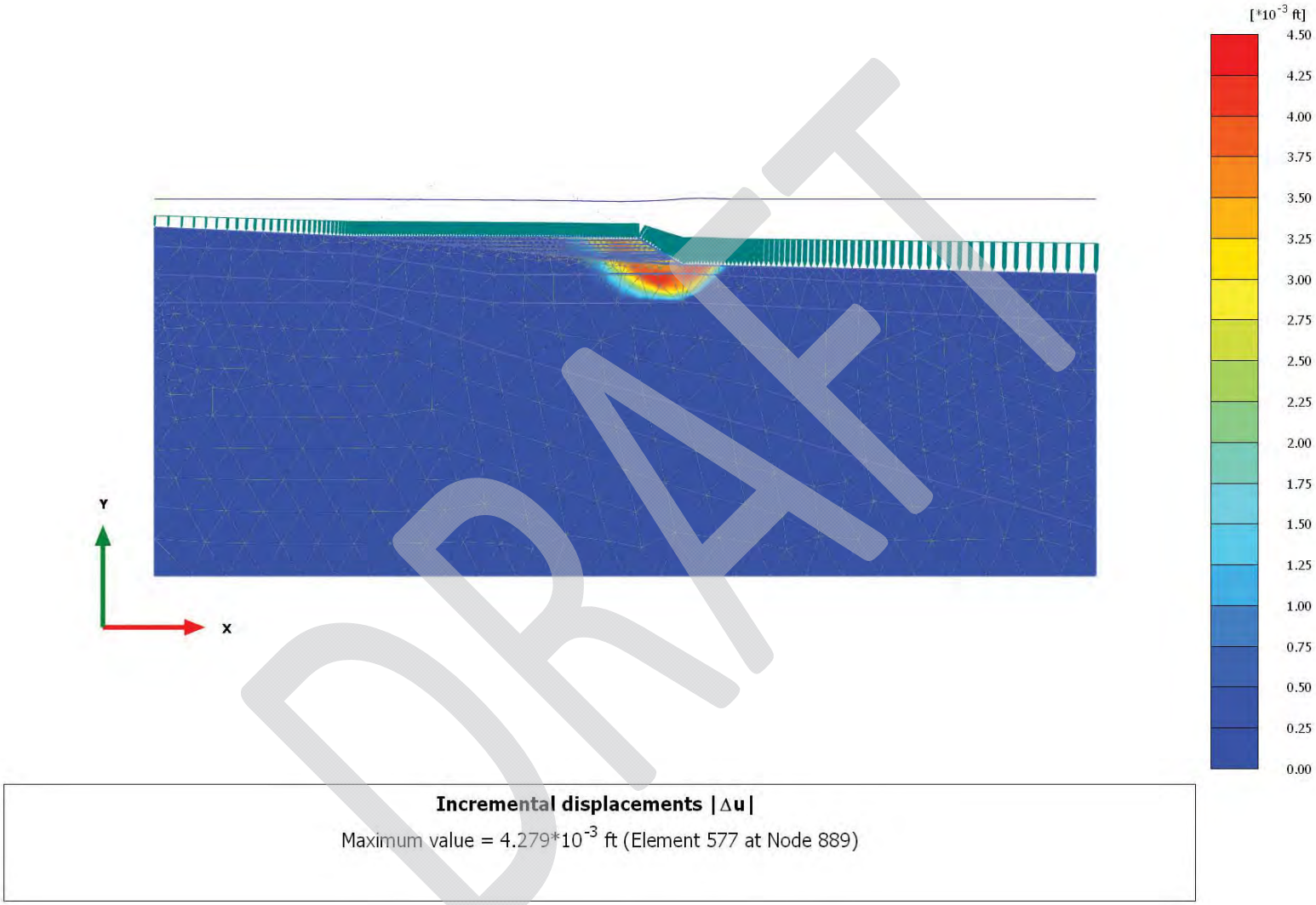

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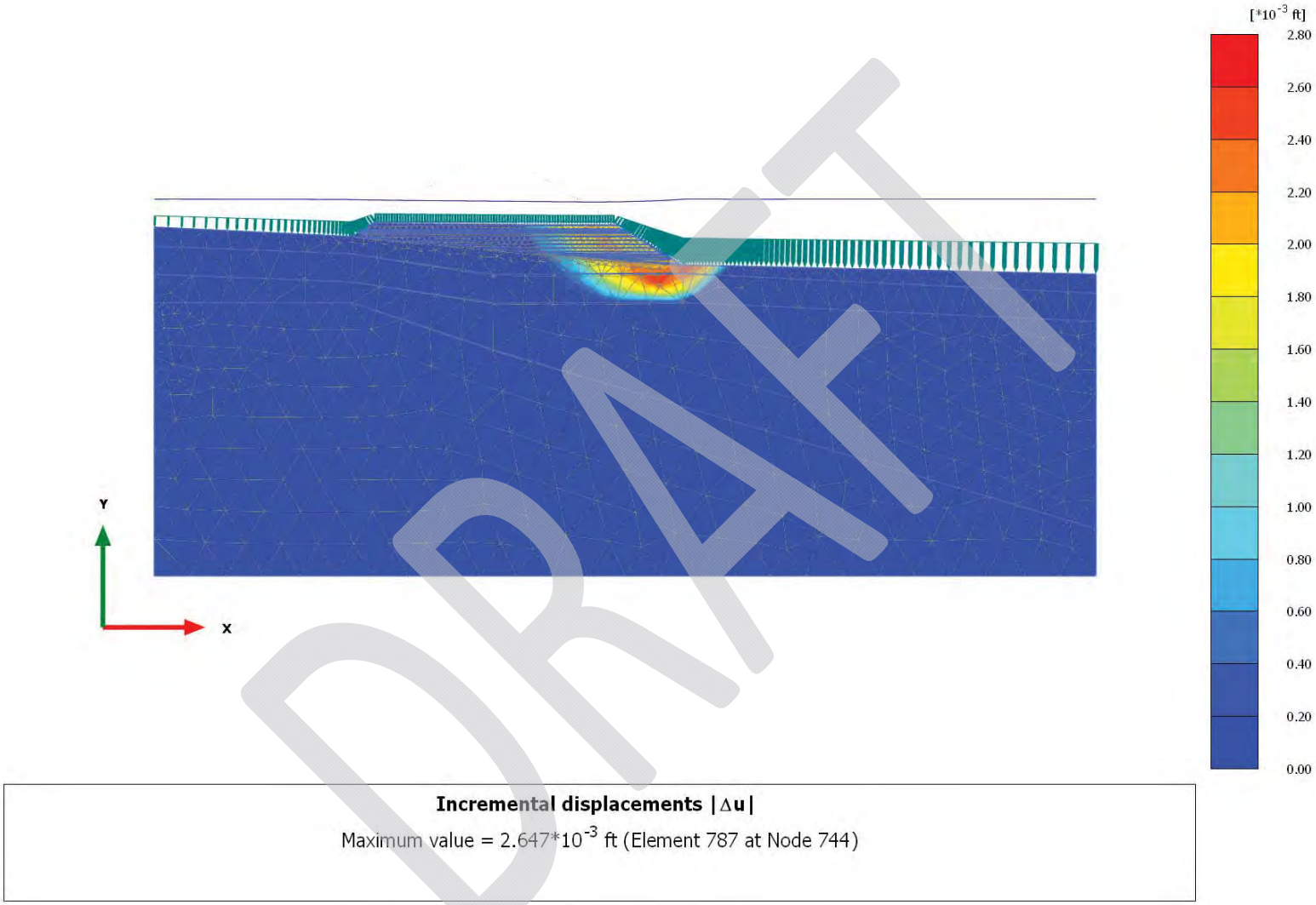
Slope Stability for Stage 1 at Elevation -20 feet for Section C-C'



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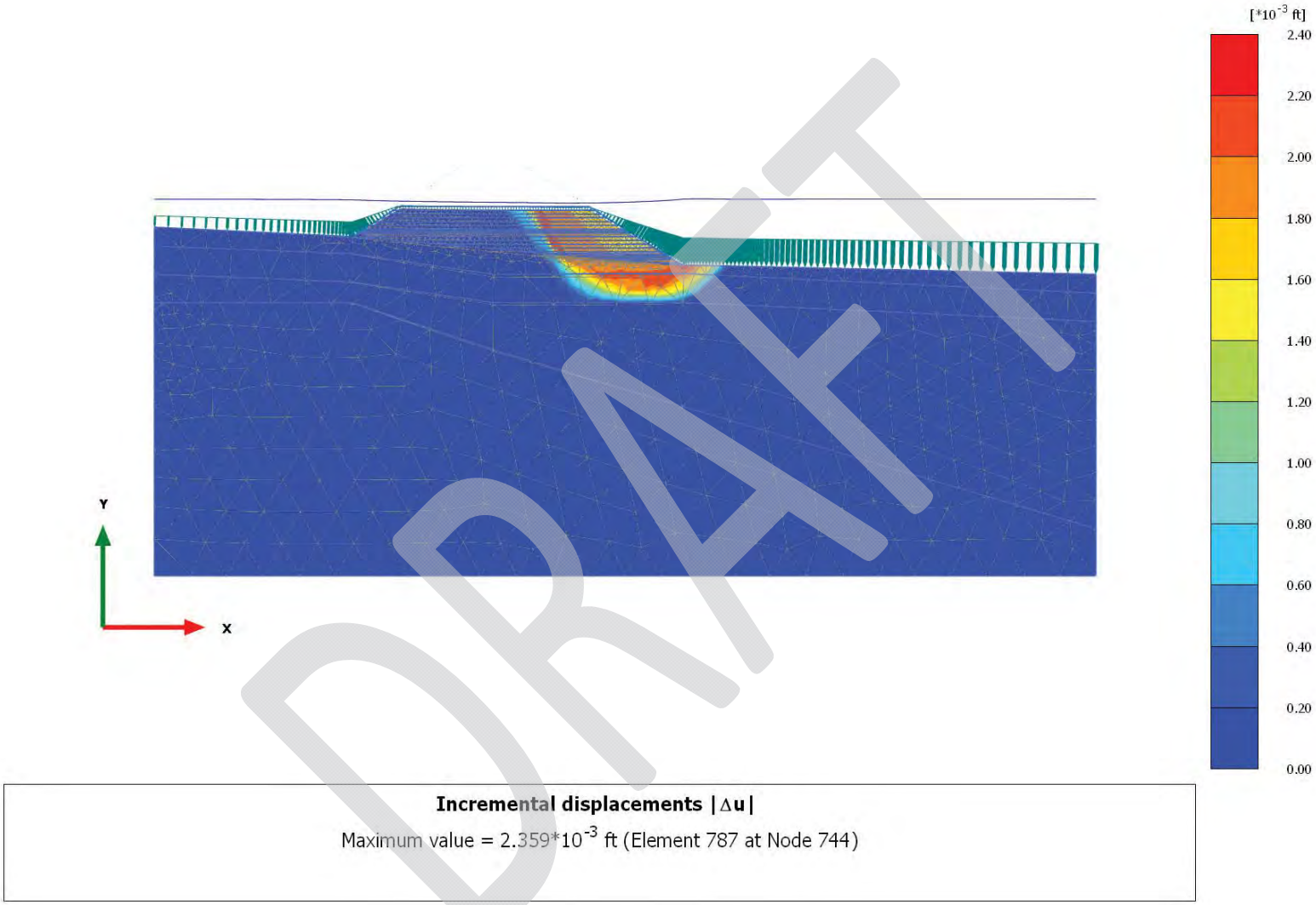
Slope Stability for Stage 2 at Elevation -12 feet for Section C-C'



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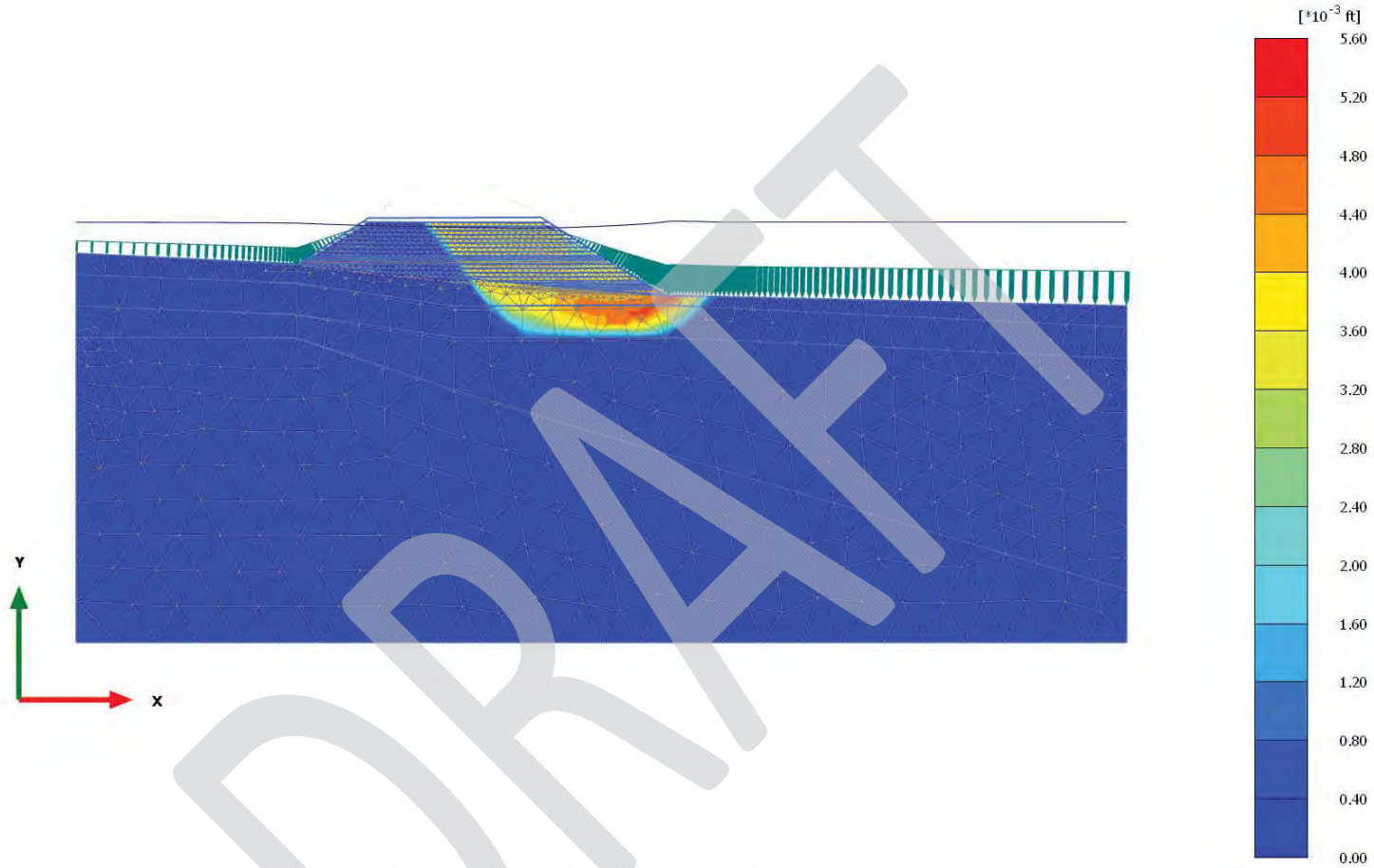
Slope Stability for Stage 3 at Elevation -4 feet for Section C-C'



Incremental displacements |Δu|

Maximum value = $5.345*10^{-3}$ ft (Element 829 at Node 728)

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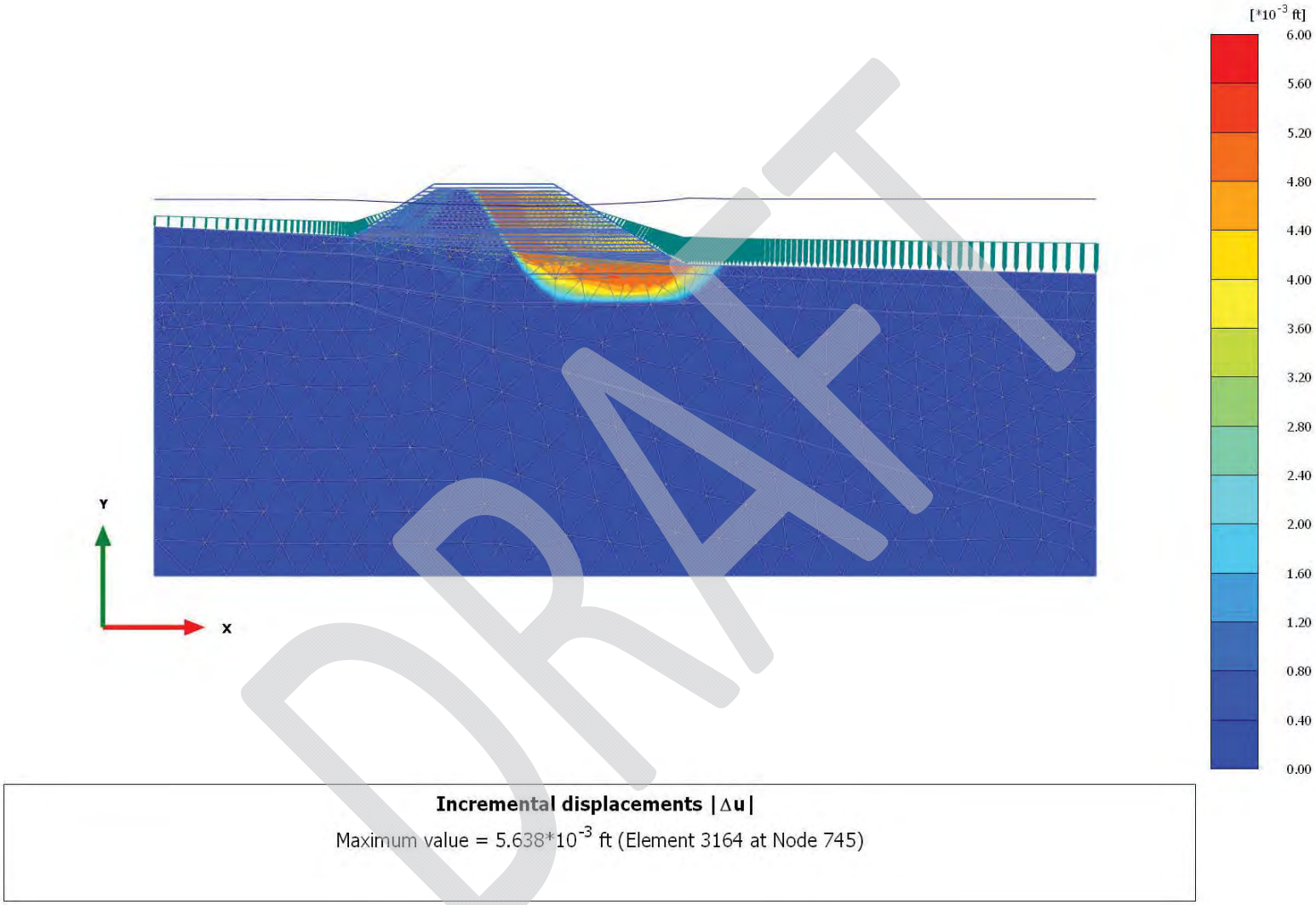
Slope Stability for Stage 4 at Elevation +2 feet for Section C-C'



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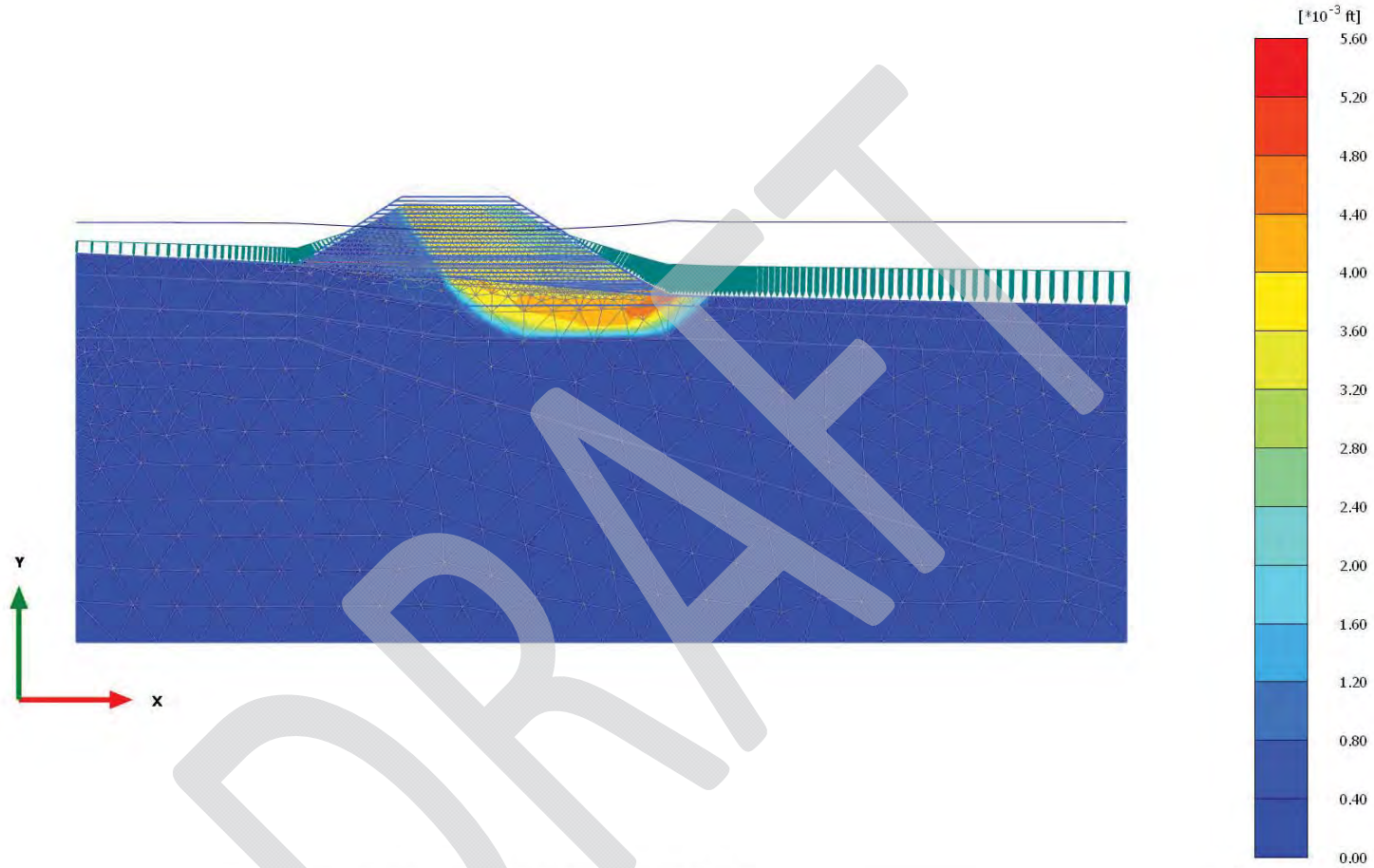
Slope Stability for Stage 5 at Elevation +8 feet for Section C-C'



Incremental displacements |Δu|

Maximum value = $5.311*10^{-3}$ ft (Element 829 at Node 728)

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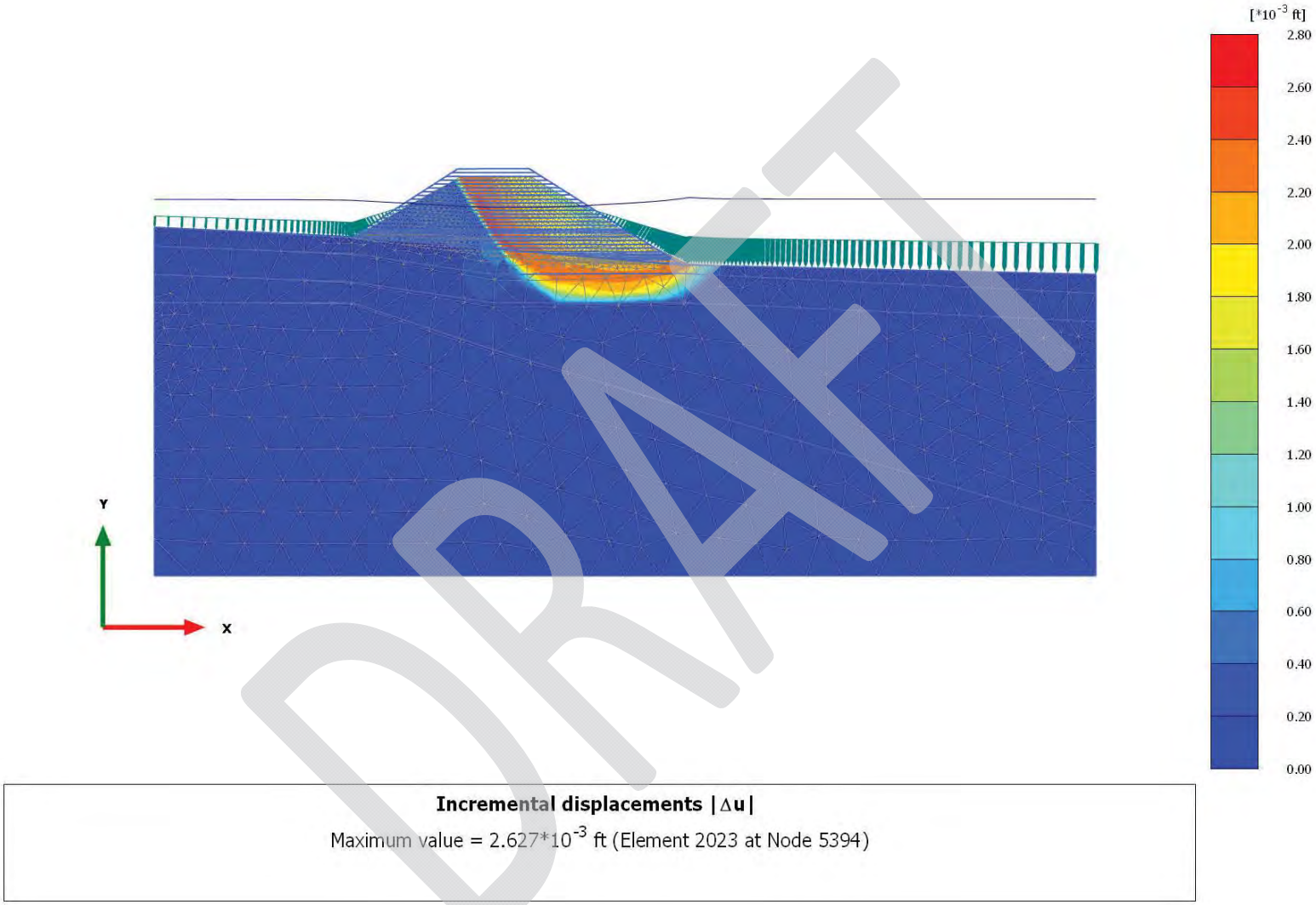
Slope Stability for Stage 6 at Elevation +12 feet for Section C-C'



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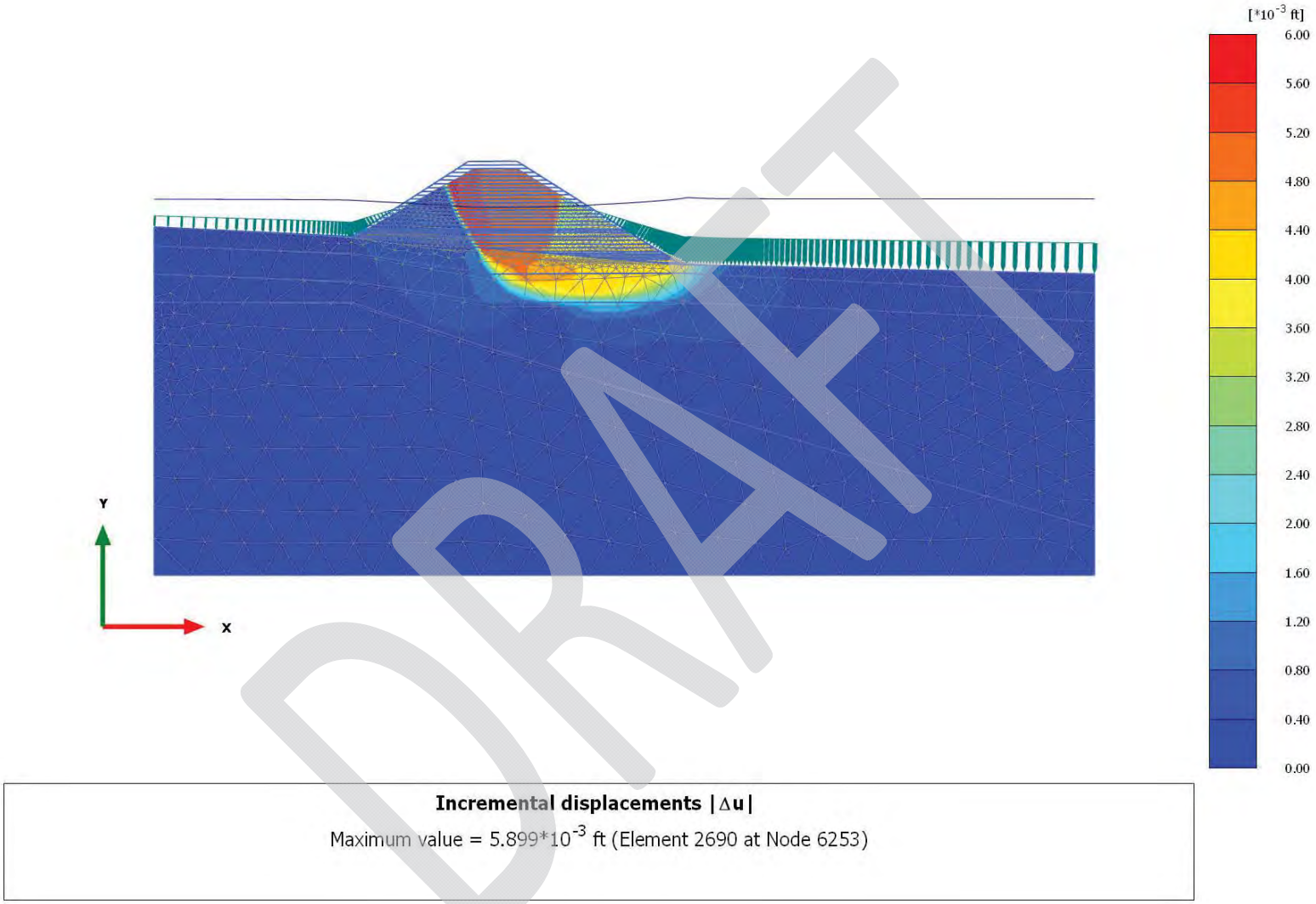
Slope Stability for Stage 7 at Elevation +16 feet for Section C-C'



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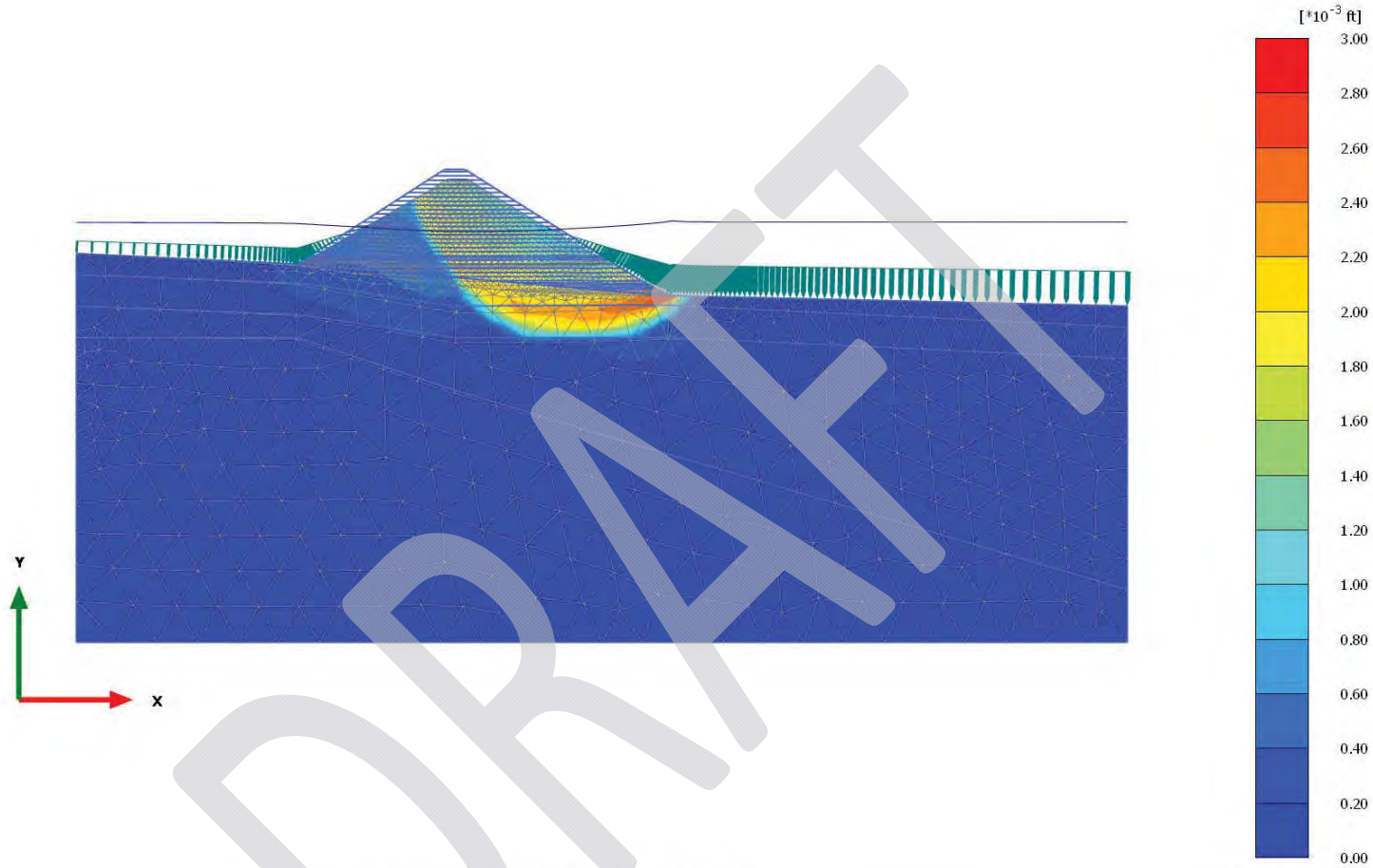
Slope Stability for Stage 8 at Elevation +20 feet for Section C-C'



Incremental displacements |Δu|

Maximum value = $2.928*10^{-3}$ ft (Element 829 at Node 728)

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Slope Stability for Stage 9 at Elevation +24 feet for Section C-C'

APPENDIX C

Seismic Deformation Analysis of Rubble-Mound Breakwater Concept

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APPENDIX C:
Seismic Deformation Analysis of Rubble-Mound Breakwater Concept

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C.2.2 Design Earthquakes 1
C.3 Results **3**
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C.1 Introduction

Newmark Sliding Block analyses were performed to estimate potential displacements of the proposed rubble-mound breakwater from earthquake-induced ground motions. The permanent deformation "Newmark" analyses completed for this study include the Simplified Rigid Block Analysis, Unified Model (flexible/rigid), and Rigorous Rigid-Block Analysis. All methods considered the OLE (Operating Level Earthquake), CLE (Contingency Level Earthquake), and MCE (Maximum Credible Earthquake) events. The calculations were performed using the USGS computer program "SLAMMER" developed by Jibson et al. (2013).

C.2 Methodology

Newmark's method treats the mobilized soil (or rock) as a rigid body that can slide plastically along a predefined slip surface. Newmark's method is best applied to translational block slides and rotational slumps" (Jibson 1993).

The Simplified Rigid Method predicts earthquake-induced displacements using the empirical relationships proposed by (Saygili and Rathje, 2008) which correlate displacement to critical (yield accelerations) and peak ground acceleration by "conducting rigorous Newmark integrations on 2,383 strong motion records for critical acceleration values between 0.05g and 0.3g"(Jibson et al., 2013) . The Unified Method computes the dynamic response and the plastic slip independently and includes additional input parameters (e.g. Site Period). The site periods and mean shaking period were calculated assuming a shear wave velocity of 600 ft/sec, a maximum slice thickness of 36.9 feet (as determined form the limit equilibrium slope stability analysis), and average moment magnitudes and earthquake distances from USGS deaggregation. The Rigorous Rigid-Block Analysis is a decoupled analysis which incorporates scaled earthquake ground motion records to estimate displacement.

C.2.1 Critical (Yield) Acceleration

Newmark sliding block analyses do not consider down slope movements that occur at accelerations less than the critical (yield) acceleration. To obtain the yield acceleration for the breakwater, limit equilibrium slope stability calculations were iterated by varying the pseudostatic horizontal coefficient of acceleration, k_h, until a factor of safety equal to unity was reached. The pseudostatic analyses to determine critical yield accelerations were completed with the computer software SLOPE/W version 8.0 (GeoStudio 2012) using the Spencer limit equilibrium method (Spencer 1967).

Critical (yield) acceleration of the rubble-mound was calculated with the following method:

- Perform slope stability analysis (in Slope/W) to identify critical failure surfaces by evaluating both rotational slump and sliding block modes of failure for static and pseudostatic conditions. The pseudostatic cases were evaluated using the Hynes-Griffin and Franklin (1984) methodology where the horizontal coefficient of acceleration is equal to ½ PGA_{rock}.
- Using the critical failure surface(s) identified by the slope stability analysis, the horizontal coefficient of acceleration was back-calculated for a factor of safety equal to unity ($F_s \approx 1$). This K_h corresponds to the yield acceleration. For conservatism, the lowest of the K_h values was used in the Newmark sliding block analyses.

C.2.2 Design Earthquakes

The 2002 United States Geological Survey (USGS) probabilistic seismic hazard deaggregations were reviewed to determine the regional source zone contribution for the MCE, OLE and CLE events. The deaggregations indicate that the seismic hazard at the site is influenced primarily by crustal strike slip faults with moment magnitudes, M_w, ranging between 5.0 and 8.2.

The contributions of each regional source zone as determined by the deaggregations are summarized in Table C.2-1 through Table C.2-3, for the OLE, CLE, and MCE events respectively.

Table C.2-1. Summary of Seismic Hazard Deaggregation for OLE Event

Source Fault	Focal Mechanism	Magnitude (M_w)	% Contribution to Seismic Hazard
Eastern Denali	Strike Slip	7.4	22%
Fair weather/ Queen Charlotte	Strike Slip	5.0-7.3	60%
Transition	Strike Slip	8.2	17%

Table C.2-2. Summary of Seismic Hazard Deaggregation for CLE Event

Source Fault	Focal Mechanism	Magnitude (M_w)	% Contribution to Seismic Hazard
Eastern Denali	Strike Slip	7.4	30%
Fair weather/ Queen Charlotte	Strike Slip	5.0-7.3	57%
Transition	Strike Slip	8.2	12%

Table C.2-3. Summary of Seismic Hazard Deaggregation for MCE Event

Source Fault	Focal Mechanism	Magnitude (M_w)	% Contribution to Seismic Hazard
Eastern Denali	Strike Slip	7.4	66%
Fair weather/ Queen Charlotte	Strike Slip	5.0-7.3	32%
Transition	Strike Slip	8.2	2%

The results of the deaggregations were used to select appropriate earthquake acceleration time histories for the OLE, MCE, and CLE ground motions from 2,100 strong-motion records from 28 earthquakes along with a search interface of the USGS software. The earthquake acceleration time histories were proportionally selected to reflect the site conditions and earthquake types that contribute to the seismic hazard at the site. The selected earthquakes were then scaled to the OLE, CLE, and MCE bedrock peak ground acceleration, PGA_{rock}, specific to our site for input into the Newmark analysis. The selected earthquake records are presented in Table C.2-4.

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Table C.2-4. Summary of Selected Earthquake Records Used in Rigorous Rigid Block Analysis

Earthquake	Year	Magnitude (M_w)	PGA	Mean Period (sec)	Epi. Dist. (mi)	Focal Dist. (mi)	Rup. Dist. (mi)	Focal Mechanism	Site Class
Kobe, Japan	1995	6.9	0.079	1.24	29.5	31.6	13.3	Strike-slip	E
Kobe, Japan	1995	6.9	0.064	1.43	29.5	31.6	13.3	Strike-slip	E
Kobe, Japan	1995	6.9	0.243	0.76	28.6	30.6	11.9	Strike-slip	E
Kobe, Japan	1995	6.9	0.212	0.73	28.6	30.6	11.9	Strike-slip	E
Kobe, Japan	1995	6.9	0.611	1.13	8.1	13.8	0.9	Strike-slip	E
Kobe, Japan	1995	6.9	0.616	0.99	8.1	13.8	0.9	Strike-slip	E
Kocaeli, Turkey	1999	7.5	0.249	0.87	69.8	70.5	43.2	Strike-slip	E
Kocaeli, Turkey	1999	7.5	0.185	0.98	69.8	70.5	43.2	Strike-slip	E
Morgan Hill, San Francisco	1984	6.2	0.046	0.52	33.7	34.2	33.5	Strike-slip	E
Morgan Hill, San Francisco	1984	6.2	0.068	0.49	33.7	34.2	33.5	Strike-slip	E

C.3 Results

The following tables, Table C.3-1 and Table C.3-2, present the slope deformations estimates using Newmark Simplified Analysis, Unified Model, and Rigorous Rigid-Block Analysis, respectively.

Table C.3-1. Summary of Displacements by Newmark Simplified (Saygili and Rathje, 2008)

	OLE (50% PE in 50 year)	CLE (10% PE in 50 years)	MCE (2% PE in 50 years)
Displacement (D_n)	0.0 in	0.1 in	8.2 in

Table C.3-2. Summary of Displacements by Newmark Unified Model (Rathje and Antonakos, 2011)

	OLE (50% PE in 50 year)	CLE (10% PE in 50 years)	MCE (2% PE in 50 years)
Critical Yield Acc.	0.145 g	0.145 g	0.145 g
Site Period	0.246 sec	0.246 sec	0.246 sec
Earthquake Magnitude	5.2 M_w	5.32 M_w	8.1 M_w
PGA (bedrock)	0.07 g	0.2 g	0.53 g
Mean Shaking Period	0.362 sec	0.371 sec	0.528 sec
Estimated Displacement	0.0 in	0.01 in	21.73 in

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Table C.3-3. Summary of Displacements by Newmark Rigorous Rigid-Block Analysis

Earthquake, Year	OLE (50% PE in 50 year) Average Displacement	CLE (10% PE in 50 years) Average Displacement	MCE (2% PE in 50 years) Average Displacement
Kobe Japan, 1995	0.0 in	0.21 in	28.4 in
Kocaleli, Turkey, 1999	0.0 in	0.25 in	24.13 in
Morgan Hill, 1984	0.0 in	0.08 in	10.17 in

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APPENDIX D

Met-Ocean Analysis

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APPENDIX D:
Met-Ocean Analysis

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D.1 Introduction

The purpose of the met-ocean analysis is to develop preliminary environmental design criteria (wind, waves, and water levels in the region) for the marine facilities at Portage Cove. The met-ocean criteria are important for evaluating wave overtopping and transmission past the breakwater, and determining environmental forces on marine infrastructure, including wind and wave and loads. Wave forces vary with the cube of the wave height, and wave transmission under a floating breakwater or wave barrier is highly sensitive to the wave period (wavelength). Therefore the wave environment at the project site needs to be carefully evaluated.

D.2 Met-Ocean Analysis

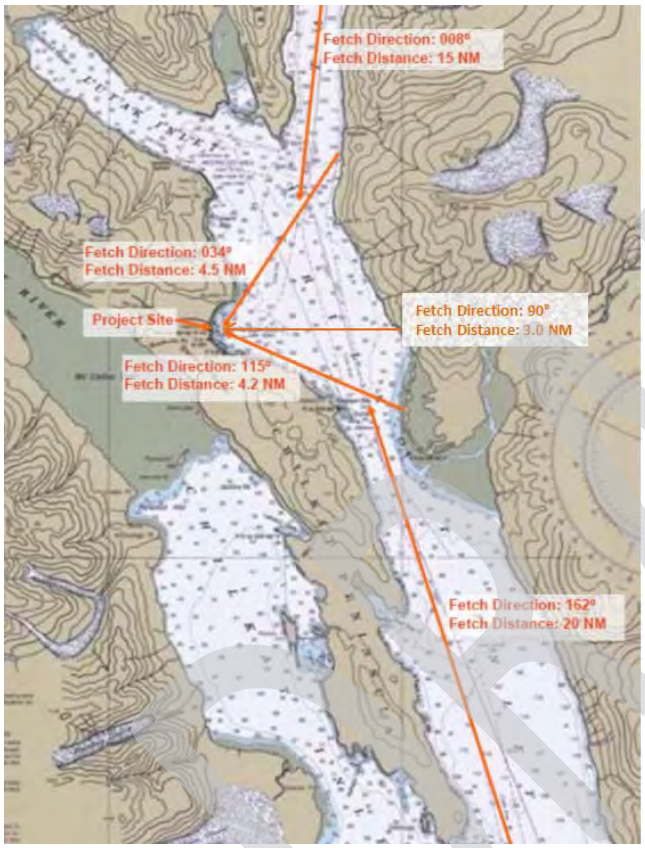

Figure D.2.1-1. Area Map – NOAA Chart No. 17317

Met-ocean design criteria have been developed based on analysis of available data and all available information, including previous reports and references. PND analyzed wind data and applied the wave model Delft3D to develop design wave criteria at the project site. The CGWAVE model was applied to evaluate wave penetration into the harbor for different breakwaters.

The project site is exposed to waves from the southeast to the northeast as shown in Figure D.2.1-1. Waves generated along a straight line fetch distance up to 4.5 nautical miles can reach the site, in addition to waves generated along much longer fetches aligned with Taiya Inlet (to the north) and Lynn Canal (to the south). Waves in Lynn Canal and Taiya Inlet are able to reach the project site by refracting and diffracting around Taiya Point and Battery Point north and south of Portage Cove.

The Deflt3D wave numerical model was used to study the generation and propagation of wind waves to the project site. The model set-up covered the entire Portage Cove and Chilkoot Inlet and extended into Taiya Inlet. The output from the Deflt3D model was then used as input to the CGWAVE numerical model to evaluate the breakwater alternatives. The numerical models are discussed in later sections of this report.

D.2.1 Criteria for Wave Conditions in a Small Boat Harbor

Recommended maximum allowable wave conditions inside marinas can vary. Typical criteria for small boat harbors allow a 1-foot wave height exceeded once per year on average, and a 2- foot wave height exceeded once every 50 years, on average. These criteria are from the "Criteria for Good Wave Climate in a Small Craft Harbor," developed by the Canadian Department of Fisheries and Oceans, Small Craft Harbors branch (Table D.2.1-1). The same criteria are also referenced in the manual "Planning and Design Guidelines for Small Craft Harbors" from the American Society of Civil Engineers.

These criteria are reasonable for use at Portage Cove. The facilities should be designed in the future so that wave conditions do not exceed the criteria in Table D.2.1-1.

Table D.2.1-1. Criteria for "Good" Wave Conditions in 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.

D.2.2 Tide and Water Levels

The tide range at Haines is large, with extreme high tides over 20 feet above Mean Lower Low Water Datum. The water elevations listed in Table D.2.2-1 are from NOAA tide information for Juneau and Haines. 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 the 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 D.2.2-1.

Table D.2.2-1. Tide and Vertical Datum

	Haines Elevation (feet, MLLW)	Juneau Elevation (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	-

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D.2.3 Wind

Wind data is available from nearby weather stations maintained by the National Climatic Data Center (NCDC). Table D.2.3-1 summarizes the data available from the nearby stations. The station locations and the corresponding wind roses are shown in Figure D.2.3-1. The wind data are assumed two minute average wind speeds. Wind direction is defined as the direction winds are travelling from.

Table D.2.3-1. Wind Data Summary

No.	Site	Date Availability Dates
1	Haines Boat Harbor	1973-1996
2	Haines Airport	1996-2013
3	Eldred Rock	1985-1989
4	Skagway	1973-2013

The wind rose for the Haines Boat Harbor indicates that winds are common form the east and the northwest as shown in the wind rose in Figure D.2.3-1. Large wind speeds up to 65 knots can also occur from the northeast to the southeast and are included in Table D.2.3-2. This wind data from the Boat Harbor station was not available from NOAA until recently. The wind data analysis in this report is generally consistent with the findings in the 2004 USACE report which used the same wind data.

The wind data extremes (directional) were analyzed to determine the wind speed associated with a given return period. Table D.2.3-2 shows the ranked wind speed for the different stations. Table D.2.3-3, Table D.2.3-4 and Table D.2.3-5 show the results of the extremal analysis. The 50-year and 2-year wind speed are summarized for the different stations in Table D.2.3-6. The data from Eldred Rock was not used for analysis due to the lack of long-term wind measurements. The return period analysis was done with all data points. The 90% confidence limits on the predictions are included.

Table D.2.3-2. Largest Recorded Wind Speeds – Filtered by Fetch Direction

Rank	Haines Boat Harbor (050°-120°)			Haines Airport Northeast (050°-120°)			Skagway (010°-030°)		
	Date	Wind Speed (Knots)	Dir (deg)	Date	Wind Speed (Knots)	Dir (deg)	Date	Wind Speed (Knots)	Dir (deg)
1	3/27/1975	65	100	5/11/1997	50	70	12/20/2012	38	30
2	6/30/1975	60	110	5/28/1998	50	60	1/25/1980	37	30
3	10/9/1991	54	100	2/3/2012	29	120	12/19/2012	37	30
4	4/3/1990	42	100	11/1/1999	28	110	1/26/1997	36	20
5	1/24/1983	38	50	10/19/1999	25	110	12/21/2008	36	20
6	1/15/1982	35	50	10/19/1998	25	110	12/18/1980	36	30
7	11/2/1984	35	50	9/7/2001	25	100	1/8/1982	35	20
8	12/19/1989	35	60	9/6/2000	24	110	2/7/1998	35	30
9	1/12/1990	35	70	2/27/2001	24	120	3/15/2003	35	30
10	12/22/1980	33	50	11/27/2002	24	110	2/1/1974	34	30

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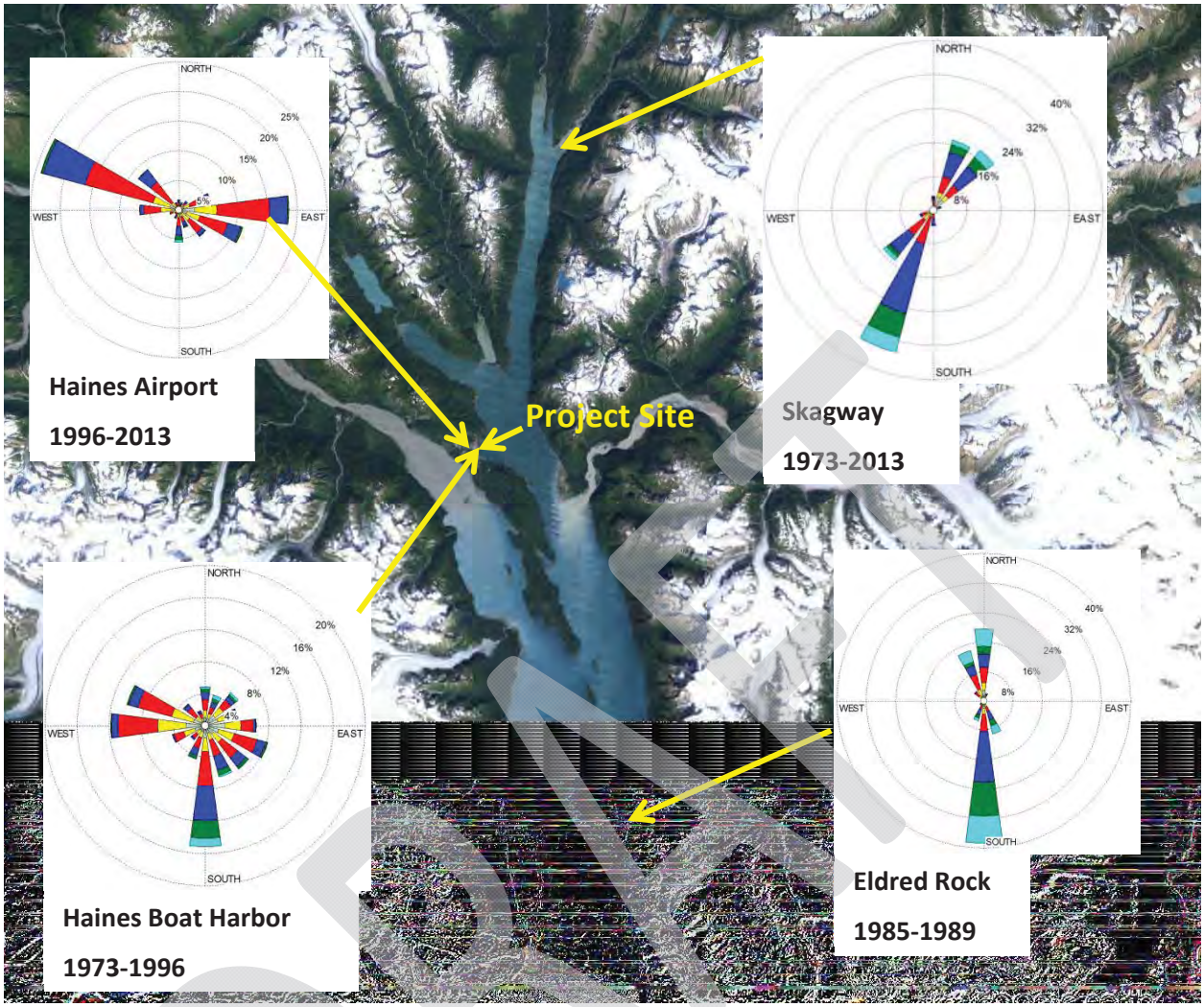

Figure D.2.3-1. Portage Cove – Wind Data Sources

Table D.2.3-3. Return Period – Haines Boat Harbor – Wind Speed (knots) – Direction: 050°-120°

N = 48; Nu = 2.00 NT = 48; K = 24	FT-1	Weibull		
Lambda = 2.00		K = 0.75	K = 1.25	K = 2.00
Correlation Coefficient	0.9328	0.9775	0.9479	0.8987
Return Period (Years)	Wind Speed (Knots)			
2	33.7	31.1	33.4	34.4
5	40.8	40.2	41.4	40.7
10	45.9	47.9	47.0	44.7

50	57.4	68.2	59.2	52.5
100	62.4	77.8	64.1	55.4
90% Confidence Interval: Return Period Wind Speed (Knots)				
5	36.1 - 45.6	30.7 - 49.6	35.8 - 47.0	36.5 - 44.9
10	39.9 - 51.9	34.7 - 61.1	40.0 - 54.0	39.8 - 49.6
50	48.6 - 66.3	45.1 - 91.3	49.0 - 69.3	46.3 - 58.6
100	52.3 - 72.4	49.9 - 105.6	52.6 - 75.6	48.7 - 62.1

Table D.2.3-4. Return Period – Haines Airport – Wind Speed (knots) – Direction: 050°-120°

N = 36; Nu = 2.00 NT = 36; K = 18	FT-1	Weibull		
Lambda = 2.00		K = 0.75	K = 1.25	K = 2.00
Correlation Coefficient	0.8323	0.9147	0.8497	0.7865
Return Period (Years)	**Wind Speed (Knots)**			
2	27.2	25.6	27.0	27.5
5	31.7	31.7	32.1	31.5
10	34.9	36.9	35.7	33.9
50	42.2	50.4	43.4	38.7
100	45.3	56.8	46.5	40.5
90% Confidence Interval: Return Period Wind Speed (Knots)				
5	27.8 - 35.6	23.9 - 39.4	27.5 - 36.7	28.0 - 34.9
10	30.0 - 39.8	26.0 - 47.7	29.9 - 41.5	29.9 - 37.9
50	34.9 - 49.5	31.3 - 69.5	35.0 - 51.8	33.6 - 43.8
100	37.0 - 53.6	33.7 - 79.8	37.0 - 56.0	35.0 - 46.1

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Table D.2.3-5. Return Period – Skagway – Wind Speed (knots) – Direction: 010°- 030°

N = 83; Nu = 2.00 NT = 83; K = 41 Lambda = 2.00	FT-1	Weibull		
		K = 0.75	K = 1.25	K = 2.00
Correlation Coefficient	0.9759	0.8648	0.9584	0.9873
Return Period (Years)	**Wind Speed (Knots)**			
2	30.4	29.0	30.2	30.9
5	33.7	32.6	33.8	34.0
10	36.1	35.6	36.4	36.0
50	41.5	43.6	41.9	39.8
100	43.8	47.4	44.1	41.2
90% Confidence Interval: Return Period Wind Speed (Knots)				
5	32.1 - 35.4	29.4 - 35.8	31.9 - 35.7	32.5 - 35.4
10	34.1 - 38.1	31.2 - 40.1	34.0 - 38.7	34.3 - 37.6
50	38.5 - 44.5	35.8 - 51.5	38.4 - 45.3	37.7 - 41.9
100	40.4 - 47.3	38.0 - 56.8	40.2 - 48.0	38.9 - 43.5

Table D.2.3-6. Return Period – Wind Speed Analysis Summary

Station - Direction	2-yr Return Period Wind Speed (Knots)	50-yr Return Period Wind Speed (Knots)
Haines Boat Harbor (050-120°)	31	68
Haines Airport (050-120°)	26	50
Skagway (010-030°)	31	40

D.2.4 Waves

The wave climate is complex with relatively large waves possible from the northeast to southeast directions. Haines residents report that waves from the southeast are most problematic for the marina due to lack of protection. In addition, waves from the northeast are reported to overtop the existing breakwater during high wave events combined with high tide. Waves (wakes) from the cruise ships can be relatively large when the ships are travelling at high speed, however cruise ship wakes are typically small in Portage Cove because of the distance to the shipping lanes and the smaller speed of the cruise ships when inside Portage Cove.

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The design wave height at the project site was estimated using both hindcast analysis and the Delft3D-Wave numerical model. The wind data from the Haines Boat Harbor was applied to the corresponding fetch directions listed in Table D.2.4-1. The results of the hindcast analysis and the Delft3D numerical models were used as inputs to the CGWAVE model to evaluate wave penetration past the breakwater alternatives.

D.2.4.1 Wave Hindcast Calculations

Fetch limited wave calculation methods were applied to determine the wave height and period associated with the wind speeds and fetch lengths shown in Figure D.2.1-1. The hindcast significant wave height (H_s), peak period (T_p), and maximum wave height (H_{max}) were calculated using standard wave prediction formulae in the U.S. Army Corps of Engineers Coastal Engineering Manual. The results are listed in Table D.2.4-1. The wave heights estimated are "deepwater", meaning in a depth offshore before they are affected by the bottom and shoal or refract. The significant wave height (H_s) is the average wave height of the one-third largest waves.

The maximum wave height is the largest single wave during a storm event and is assumed equal to 1.7 times the significant wave height. The wind speed analysis for hindcast calculations was directional, meaning the return period winds aligned with the associated fetch direction were used to calculate the return period wind speed.

The 50-year return period significant wave height at the project site is approximately 6.8 feet for winds blowing from the southeast along a straight line fetch of 4.5 nautical miles. The 2-year return period significant wave height is 2.6 feet and is used for assessing operating conditions in the harbor, rather than design loads on the breakwater and harbor structures.

Table D.2.4-1. Hindcast Analysis – Portage Cove – Wave Height

No.	Direction - Fetch	Wind Speed (Knots)	H_s (Feet)	H_{max} (Feet)	T_p (s)
2-Year Return Period					
1	Northeast – 4.5 NM	31	2.6	4.4	2.5
2	East – 3.0 NM	31	2.1	3.6	2.2
3	Southeast – 4.2 NM	31	2.5	4.3	2.4
50-Year Return Period					
4	Northeast – 4.5 NM	68	6.8	11.6	3.5
5	East – 3.0 NM	68	5.6	9.5	3.0
6	Southeast – 4.2 NM	68	6.6	11.2	3.4

D.2.4.2 Delft3D-Wave Numerical Model

The Delft3D-Wave numerical model was applied to simulate the generation and transformation of wind-generated waves. The model computes the non-steady propagation of short crested waves over an uneven bottom, considering: wind action; energy dissipation due to bottom friction; wave breaking; refraction (due to bottom topography, water levels and flow fields); shoaling and directional spreading. The program is widely used and is based on the spectral model SWAN, developed at Delft University of Technology. The numerical model results are likely more accurate than the straight line fetch formula for wind-wave growth applied in the previous section.

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Bathymetry data obtained from the NOAA national Geophysical Data Center (NGDC) is shown in Figure D.2.4-1 and was used to develop the model grids for the project site. The model domain (Figure D.2.4-2) covers Portage Cove, Chilkoot Inlet, Taiya Inlet and Lynn Canal. A smaller grid is nested inside the larger grid. The nested grid is more refined, with closing spacing of the grid nodes. The observation locations (model output points) are shown in Figure D.2.4-3.

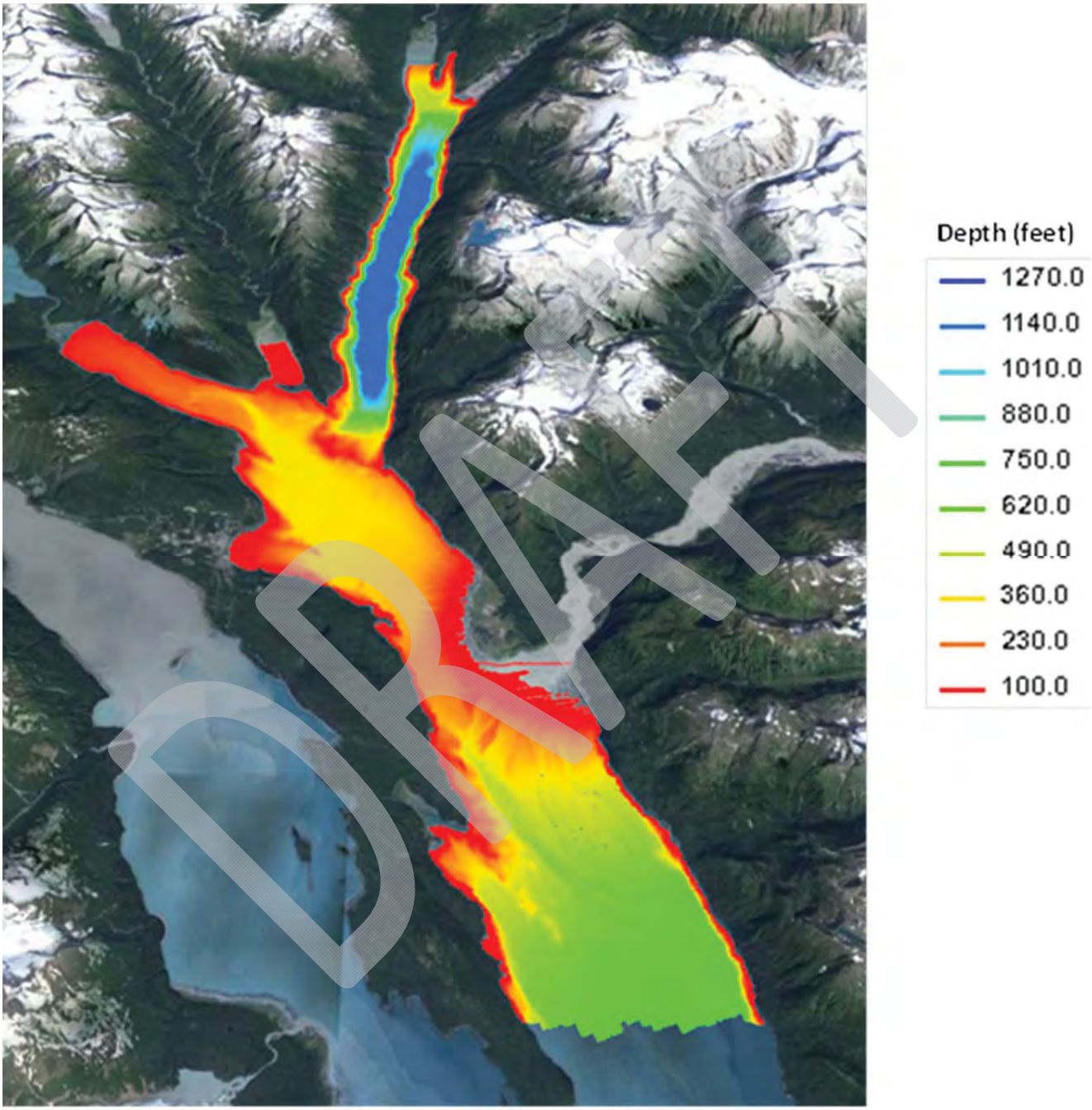

Figure D.2.4-1. Delft-3D Numerical Model – Bathymetry

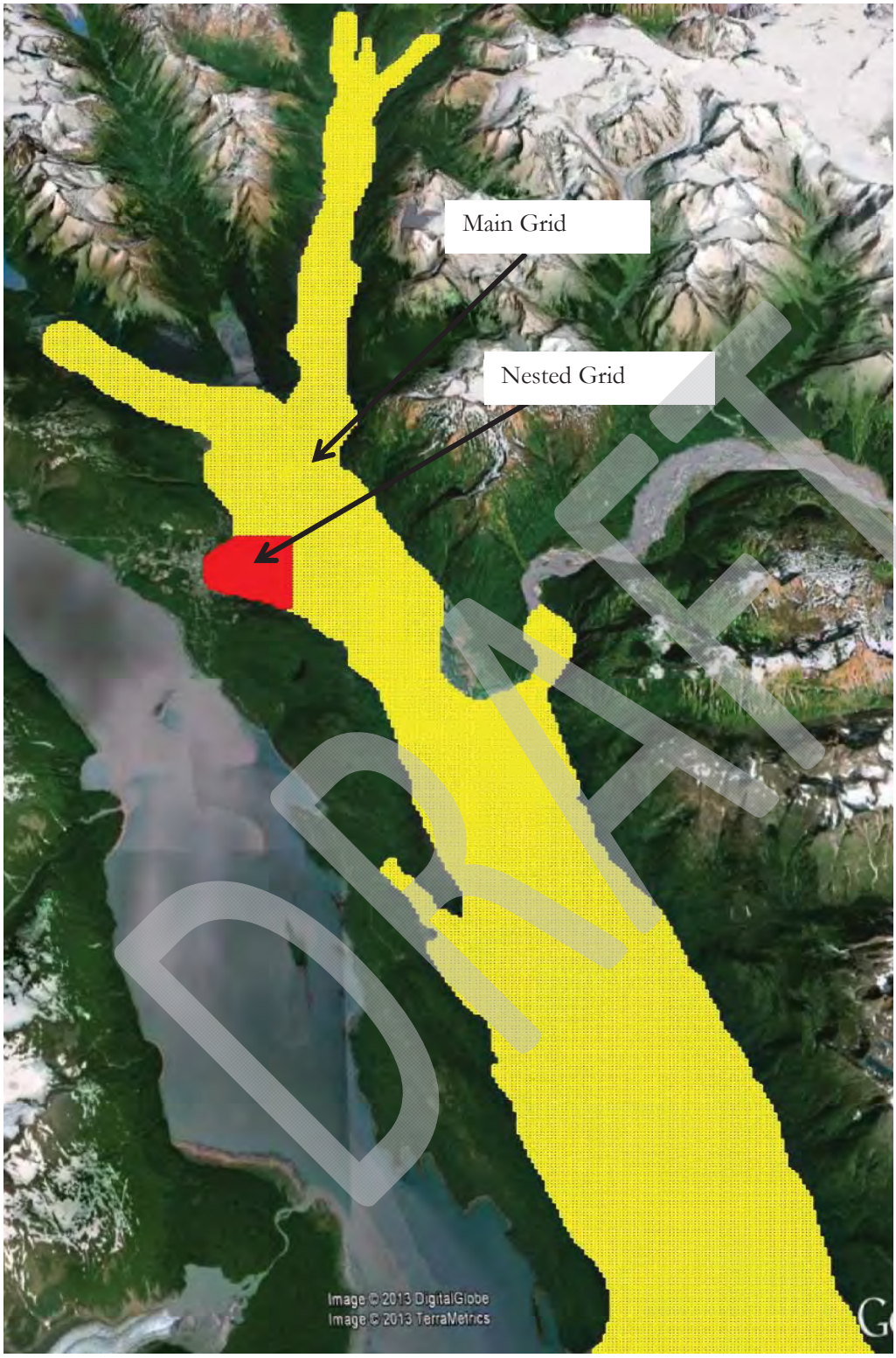

Figure D.2.4-2. Delft-3D Numerical Model Domain

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Figure D.2.4-3. Delft3D Numerical Model Observation Locations

The maximum significant wave height is 6.9 feet for a 50-year return period wind speed blowing from the east. The waves from the southeast and northeast are slightly smaller, though the fetch distance is relatively longer. The waves experience diffraction effects before reaching the project site that lead to smaller wave heights. The maximum significant wave height is 6.3 feet for a wind speed of 68 knots blowing from the southeast and 6.5 feet for the same wind blowing from the northeast. The model simulations show no unexpected results and are mostly comparable to the hindcast analysis. However, the model predicts larger wave heights for the winds blowing from the east. The model results are also consistent with the previous wave study performed by the USACE at the project site and with local observations. The output from the Delft3D model was used as input to the CGWAVE model to evaluate the different breakwater alternatives.

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Table D.2.4-2. Delft3D Numerical Model – Input and Output

Run ID	Wind		Output					
			Point 1 (offshore)			Point 2 (near breakwater)		
	Spd (Knots)	Dir (Deg)	Hs (Ft)	Tp (s)	Dir (Deg)	Hs (Ft)	Tp (s)	Dir (Deg)
1	32	10	3.4	3.5	37	2.9	3.2	49
2	40	10	3.6	3.5	36	3.0	3.2	46
3	68	50	7.4	4.4	59	6.5	4.3	69
4	68	90	7.3	4.4	89	6.9	4.4	89
5	68	120	6.9	4.4	103	6.3	4.3	100
6	30	130	2.4	3.0	108	2.3	3.0	104
7	60	130	5.8	4.1	108	5.4	4.0	104

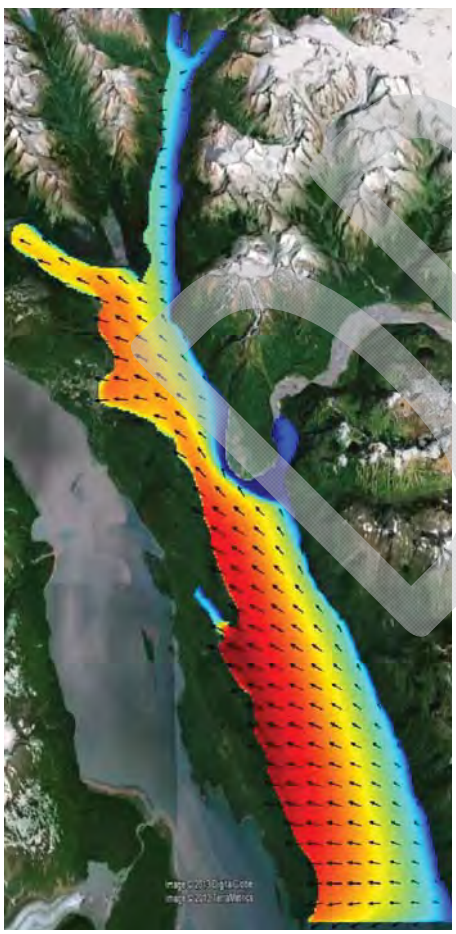

Figure D.2.4-4. Delft3D Numerical Model Output – Run 4 (Input: Wind 68 knots, Direction 90°)

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D.2.4.3 CGWAVE Numerical Model

CGWAVE is a general purpose numerical (computer) wave prediction model. It is suitable for predicting wave fields propagating into harbors, and around fixed or floating structures. The model includes the combined effects of wave refraction and diffraction based on solutions to the depth averaged mild-slope equation. The model also includes the effects of reflection, diffraction, shoaling and wave dissipation by bottom friction and wave breaking. The model is described in Demirbilek and Panchang (1998). The model is phase-revolving, meaning individual wave crests and troughs are seen as they move into a harbor.

CGWAVE was used to study the transformation of waves around the proposed breakwater alternatives. CGWAVE can model irregular waves (waves with different periods combined) through the principle of linear superposition. However, a regular wave train was applied to simplify interpretation of model results.

Bathymetry data from NOAA (shown in Figure D.2.4-1) and PND survey in May 2013 was combined to develop the model bathymetry, Figure D.2.4-5. The model domain extends 2,000 feet from the coastline. The shoreline and the rubble mound breakwater were modeled as fully absorbing boundaries. A reflection and transmission coefficient was input for the wave barrier and floating breakwater. The inputs used at the sea boundary are based on the output from the Delft3D wave model, listed in Table D.2.4-3.

Wave height outputs were obtained at four different locations, shown in Figure D.2.4-6 and summarized in Table D.2.4-3. Model output examples are shown in Figure D.2.4-7 – Figure D.2.4-11. Digital animation files also show the wave transformation patterns and are attached as separate files. A screenshot from the animation is shown in Figure D.2.4-11.

The wave height at the four observation locations are compared for the different alternatives in Figure D.2.4-12. The best protection is offered by the rubble mound breakwater. The maximum wave height at Point 1 is 0.1 feet for the 50-year wave from the southeast. The wave heights inside the marina are well below the design criteria limit for all breakwater alternatives for all conditions. Other factors such as cost, function and permitting should be considered in the determining the most suitable alternative.

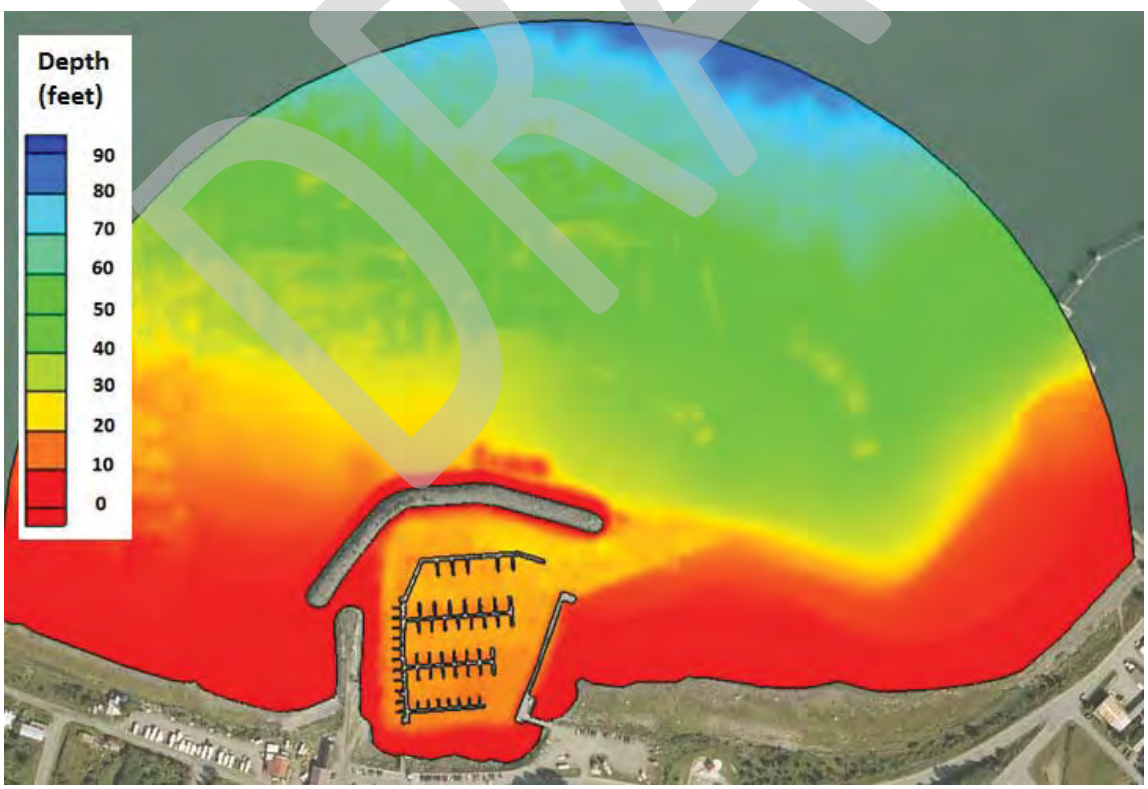

Figure D.2.4-5. CGWAVE Model Domain and Bathymetry

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Table D.2.4-3. CGWAVE Model – Input/Output

No.	Description	Input Wave			Output Wave Height			
		Hs (ft)	Tp (s)	Dir (deg)	Point 1	Point 2	Point 3	Point 4
50-yr Return Period								
Existing								
1	Northeast	6.5	4.3	050°	2.1	1.2	0.2	0.1
2	East	6.9	4.4	090°	2.8	2.3	0.3	0.1
3	Southeast	6.3	4.3	120°	2.8	2.4	0.8	0.2
Rubble Mound Breakwater								
4	Northeast	6.5	4.3	050°	0.1	0.0	0.0	0.0
5	East	6.9	4.4	090°	0.2	0.1	0.0	0.0
6	Southeast	6.3	4.3	120°	0.1	0.1	0.0	0.0
Wave Barrier								
7	Northeast	6.5	4.3	050°	0.6	0.4	0.0	0.0
8	East	6.9	4.4	090°	0.4	0.3	0.0	0.0
9	Southeast	6.3	4.3	120°	0.3	0.2	0.0	0.0
Offshore Floating Breakwater								
10	Northeast	6.5	4.3	050°	0.1	0.1	0.0	0.0
11	East	6.9	4.4	090°	0.2	0.1	0.0	0.0
12	Southeast	6.3	4.3	120°	0.4	0.1	0.0	0.0

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Figure D.2.4-6. CGWAVE Numerical Model Observation Locations

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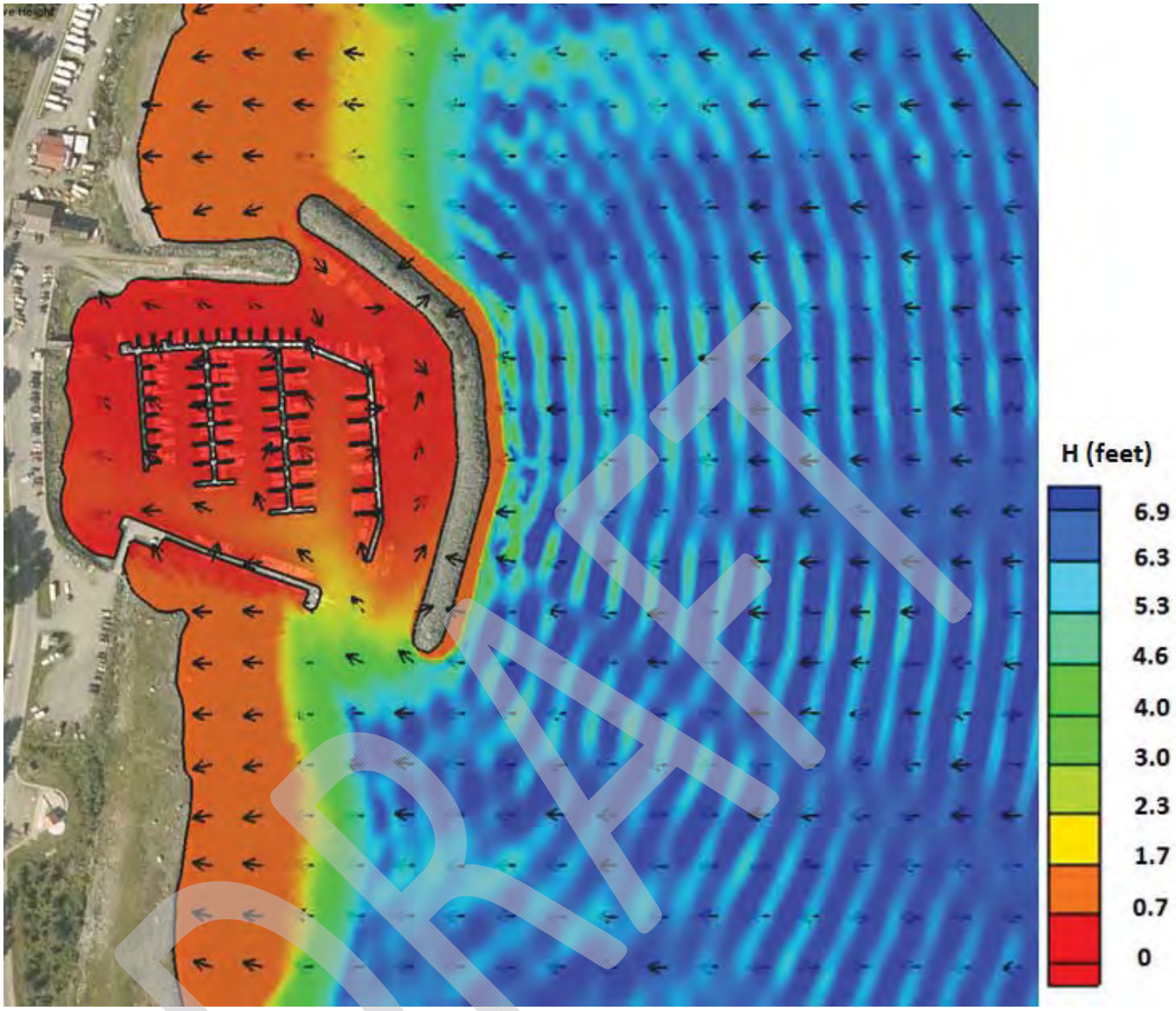

Figure D.2.4-7. CGWAVE Model Output – Run 2 (Existing)
Wave Input: H=6.9 ft, T=4.4s, Dir=090°

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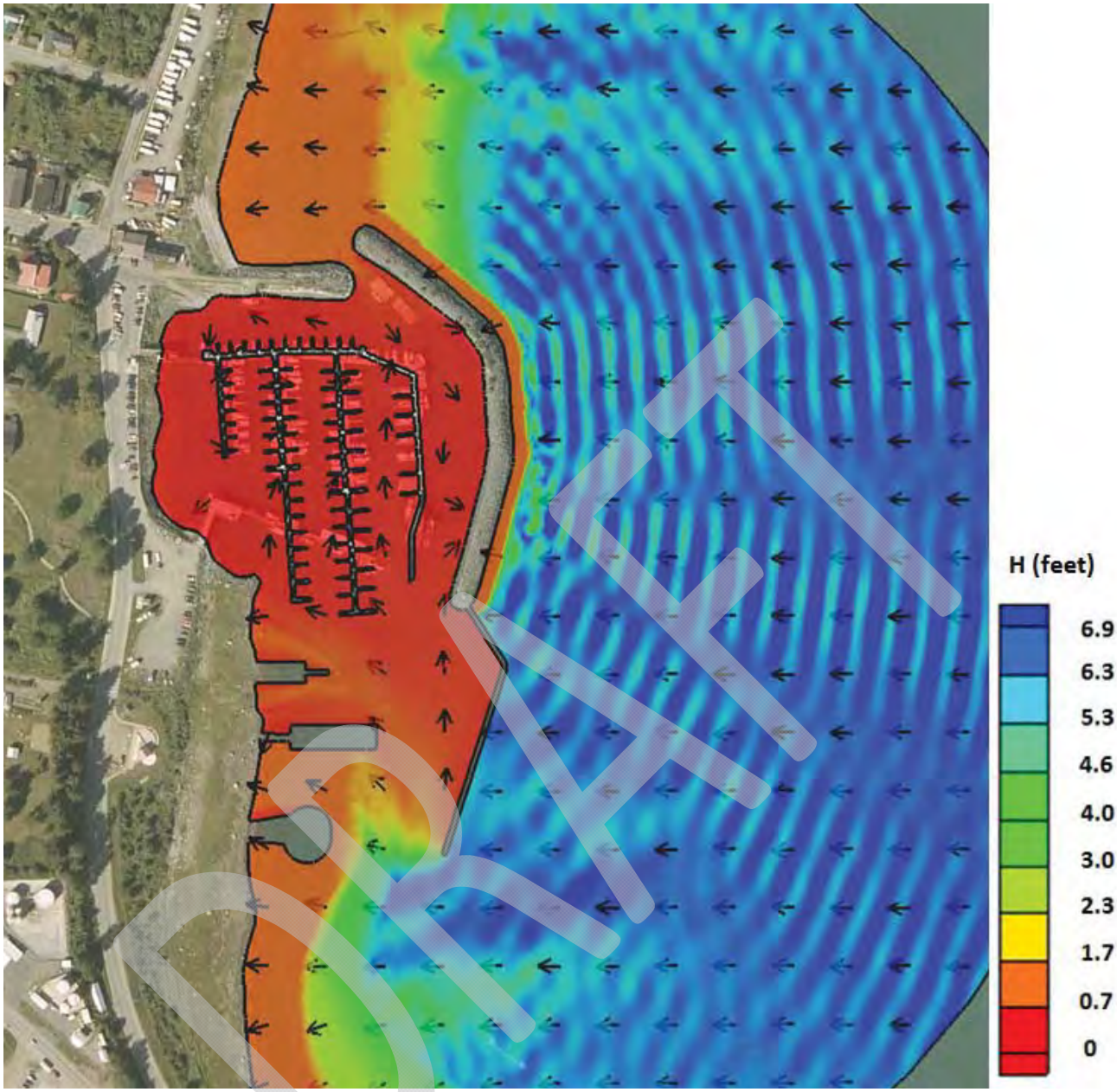

Figure D.2.4-8. CGWAVE Model Output – Run 5 (Rubble Mound Breakwater)
Wave Input: H=6.9 ft, T=4.4s, Dir=090°

(The breakwater was modeled as fully absorbing. The model shows the breakwater extents visible above the water).

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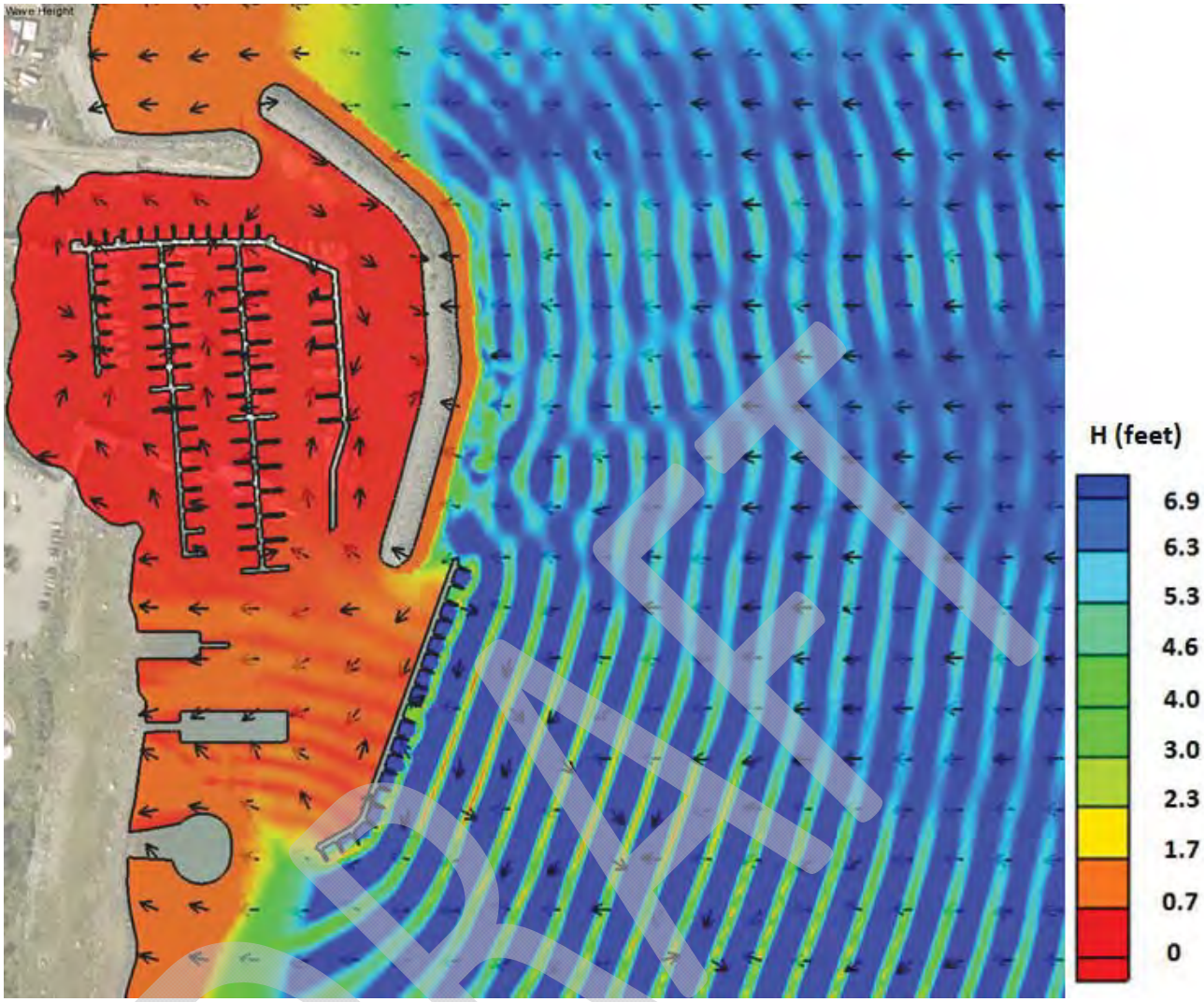

Figure D.2.4-9. CGWAVE Model Output – Run 8 (Wave Barrier)
Wave Input: H=6.9 ft, T=4.4s, Dir=090°

(The wave barrier was modeled as fully reflecting with an assumed transmission coefficient of 0.6).

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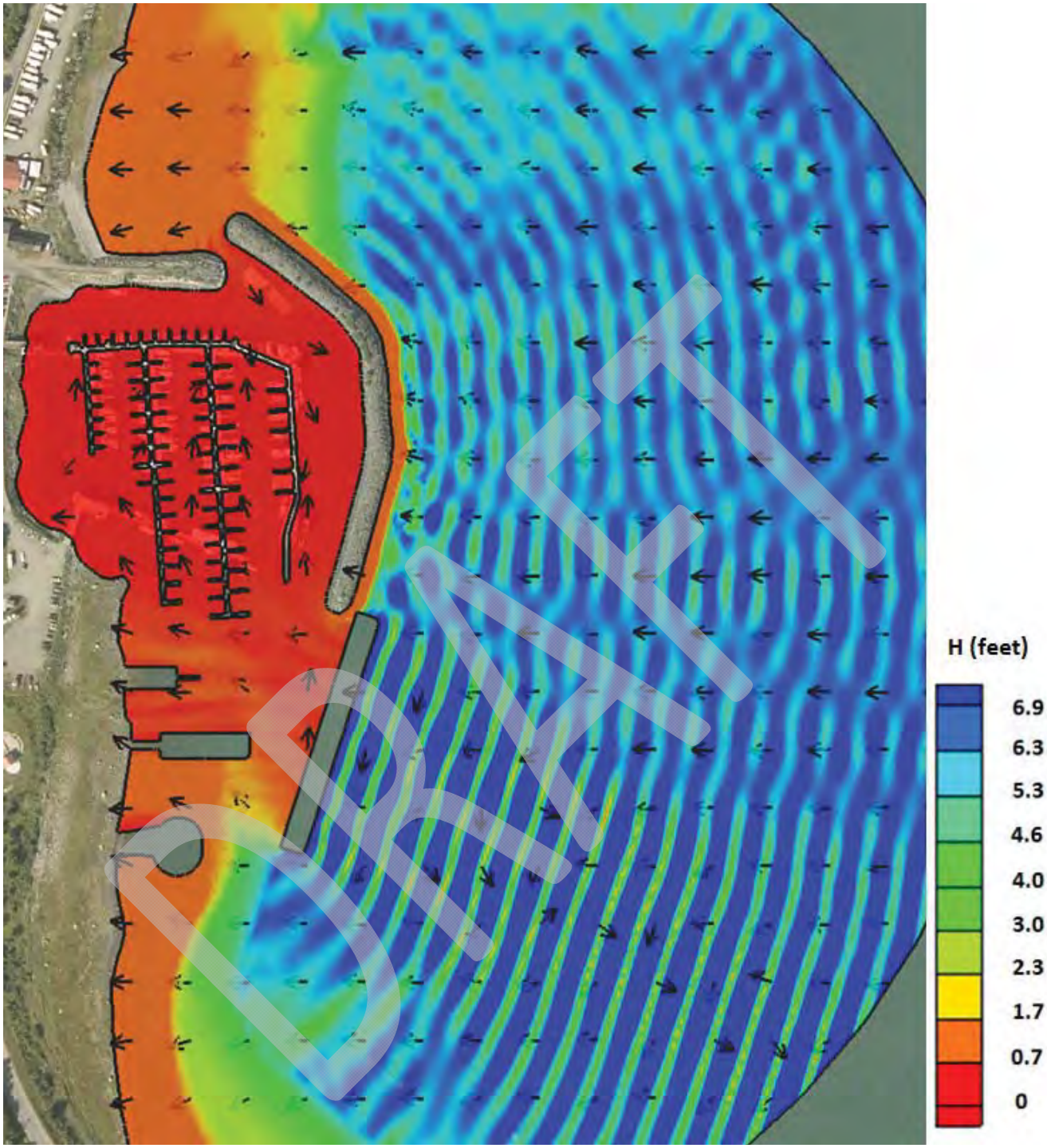

Figure D.2.4-10. CGWAVE Model Output – Run 11 (Floating Breakwater)
Wave Input: H=6.9 ft, T=4.4s, Dir=090°

(The wave floating was modeled as fully reflecting with a transmission coefficient of 0.3).

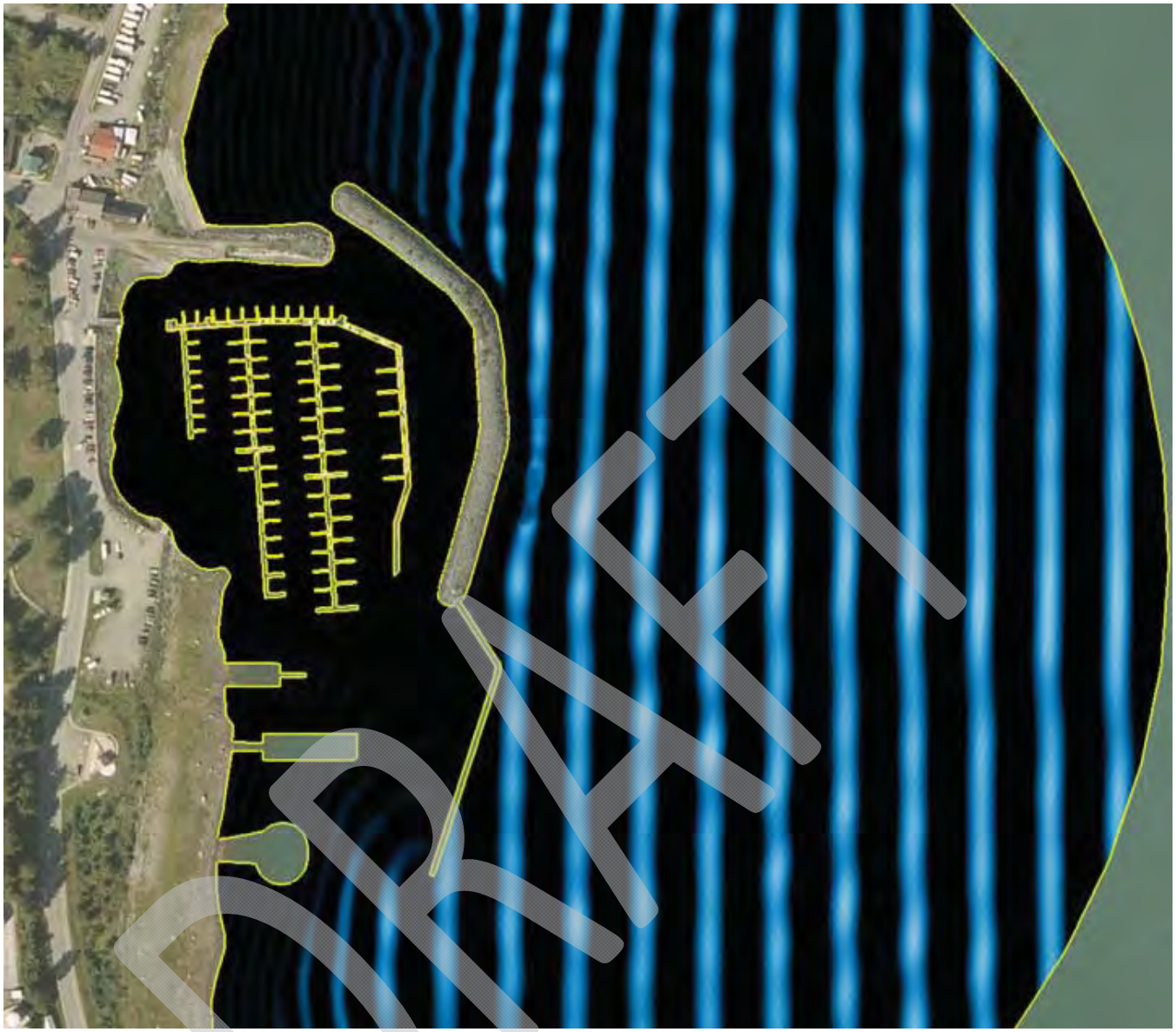

Figure D.2.4-11. CGWAVE Numerical Model Output
Screenshot from animation showing wave transformation

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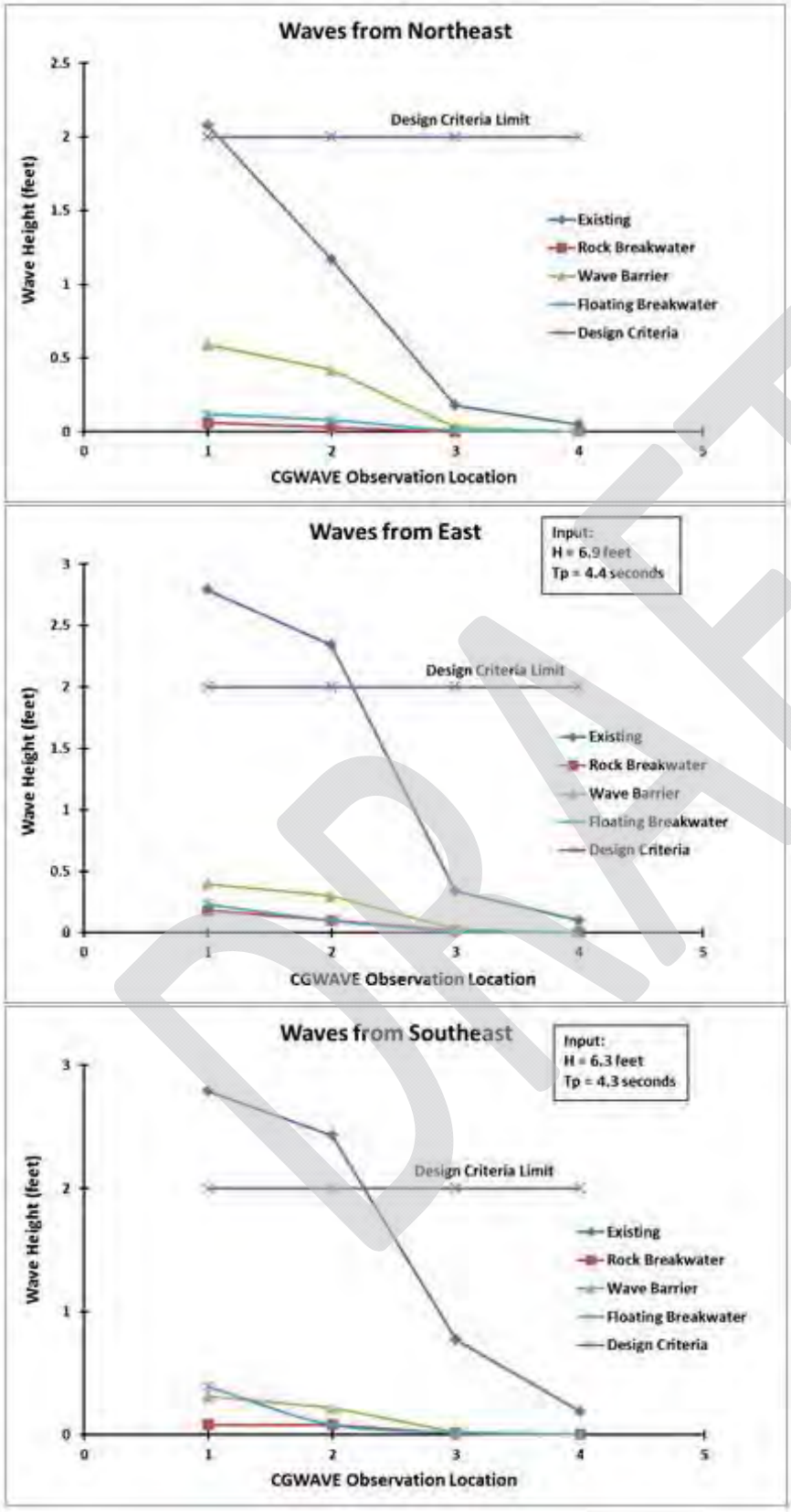

Design Criteria based on "Planning and Design Guidelines for Small Craft Harbors" from the American Society of Civil Engineers.

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Figure D.2.4-12. Breakwater Alternatives – Wave Attenuation Comparison

D.3 Floating Breakwater Analysis

This section reviews analysis methods for predicting wave transmission past a floating breakwater. The methods typically assume a rectangular breakwater cross-section. Included is a review of past studies of floating breakwaters which provide guidance on float dimensions and design.

D.3.1 Review of Previous Studies

Power Transmission Theory by Wiegel for a Thin Barrier

Theories of wave transmission past a barrier date back to at least Ursell (1947) and Weigel (1959). However, simple and reliable calculation methods for wave transmission past floating structures are not available. Table D.2.4-2 presents a summary of the wave transmission coefficients for the Portage Cove floating breakwater based on the different desktop calculation methods discussed below.

Experiments performed (Figure D.3.1-1) showed a trend of decreasing transmission coefficient with increasing wave steepness based on the laboratory measurements.

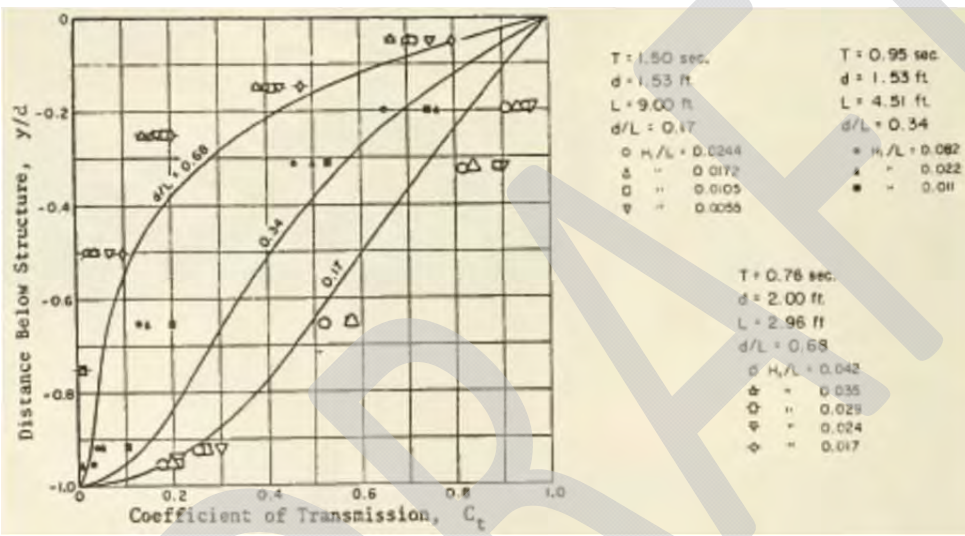

Figure D.3.1-1. Effect of Depth of Submergence of Thin, Rigid, Fixed Vertical Plate on Transmission Coefficient and Power Transmission Theory
Experimental results from Weigel (1959) (Source: Hales 1981)

The formula in equation 1 from Macagno (1953) predicts the wave transmission coefficient for a rigid structure of finite width, height and draft at a fixed position relative to the bottom. It was assumed that no overtopping took place and the dimension (height – draft of breakwater) is very large.

Equation 1

$$K_T = \frac{1}{\sqrt{1+\left(\frac{\pi B \sinh\left(\frac{2\pi d}{L}\right)}{L\cosh\left(\frac{2\pi(d-D)}{L}\right)}\right)^2}}$$

A review of wave transmission studies by PIANC (1994) indicates that this model is inadequate for important values of the relative draft D/d. For example, the transmission coefficient must be zero if the draft of the breakwater is equal to the water depth. However, the model fails to predict this.

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Stoker (1957) found the following expression for the case of a rigid board of length 2B, which is fixed at the still water surface in shallow water,

Equation 2

$$K_T = \frac{1}{\sqrt{1+\left(\frac{2\pi B}{L}\right)^2}}$$

Carr, Healy and Stelzriede (1950) developed a similar expression for shallow water as shown in equations 3 and 4. S is the ratio of the depth of the immersed body below the still water level (D) to the vertical distance from the bottom of the body to the sea bed (d-D).

Equation 3

$$K_T = \frac{1}{\sqrt{1+\left(\frac{2\pi B}{L}(1+S)\right)^2}}$$

Equation 4

$$S = \frac{D}{d-D}$$

Laboratory Experiments

Tolba (1999) performed experiments with rigid floating breakwaters to study the effect of the breadth and draft on the transmission coefficients. Figure D.3.1-2 and Figure D.3.1-3 show the results of the experiments. It can be observed that with increasing draft and breadth the transmission coefficient decreases. The influence of the wave steepness on the transmission coefficient was also investigated as shown in Figure D.3.1-4. The wave steepness has almost no influence on the transmission coefficient.

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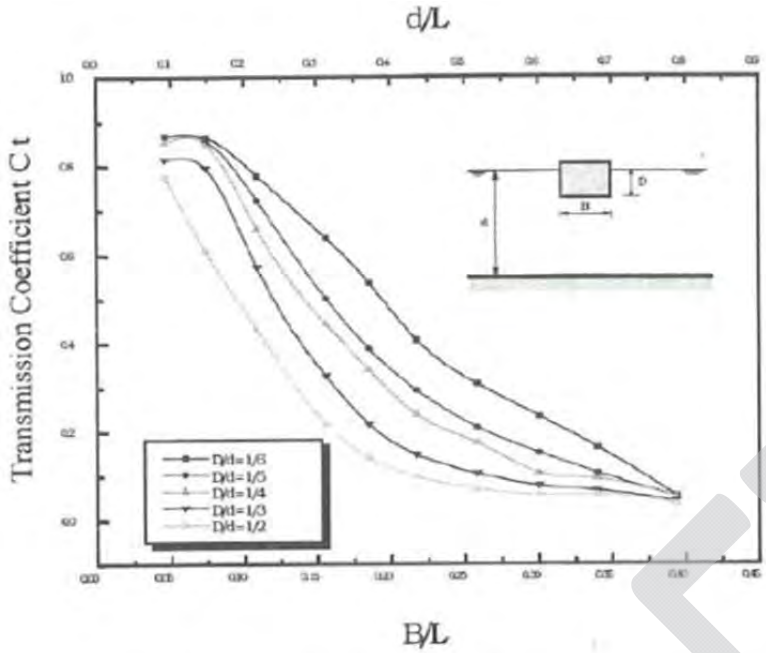

Figure D.3.1-2. Effect of Increasing Breadth on Transmission Coefficient (Source: Tolba (1999))

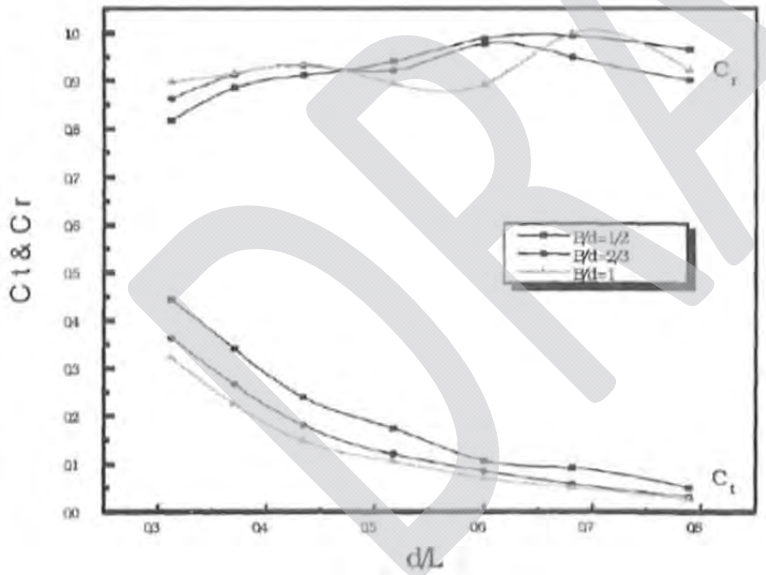

Figure D.3.1-3. Effect of Increasing Draft on Transmission Coefficient (Source: Tolba (1999))

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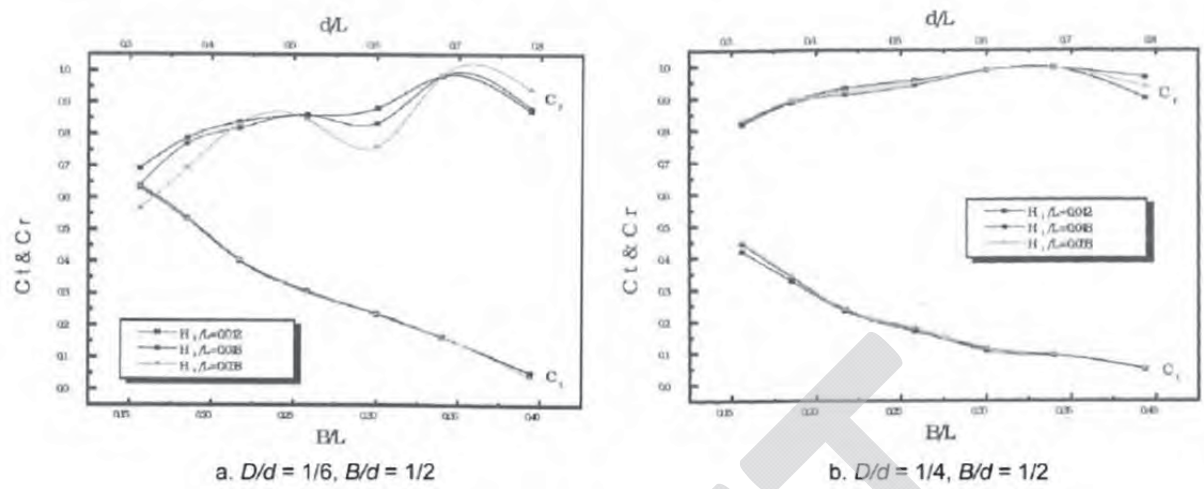

Figure D.3.1-4. Effect of Wave Steepness on Transmission Coefficient (Source: Tolba (1999))

Koutandos et al. (2005) performed similar tests on fixed rectangular floating breakwater under regular and irregular waves covering a range of shallow and intermediate waters. The tested ratios of depth to wave length where 0.04< d/L <0.35. Figure D.3.1-5 and Figure D.3.1-6 summarize the results of the tests. The regular and irregular wave forcing follow similar trends as observed from the results.

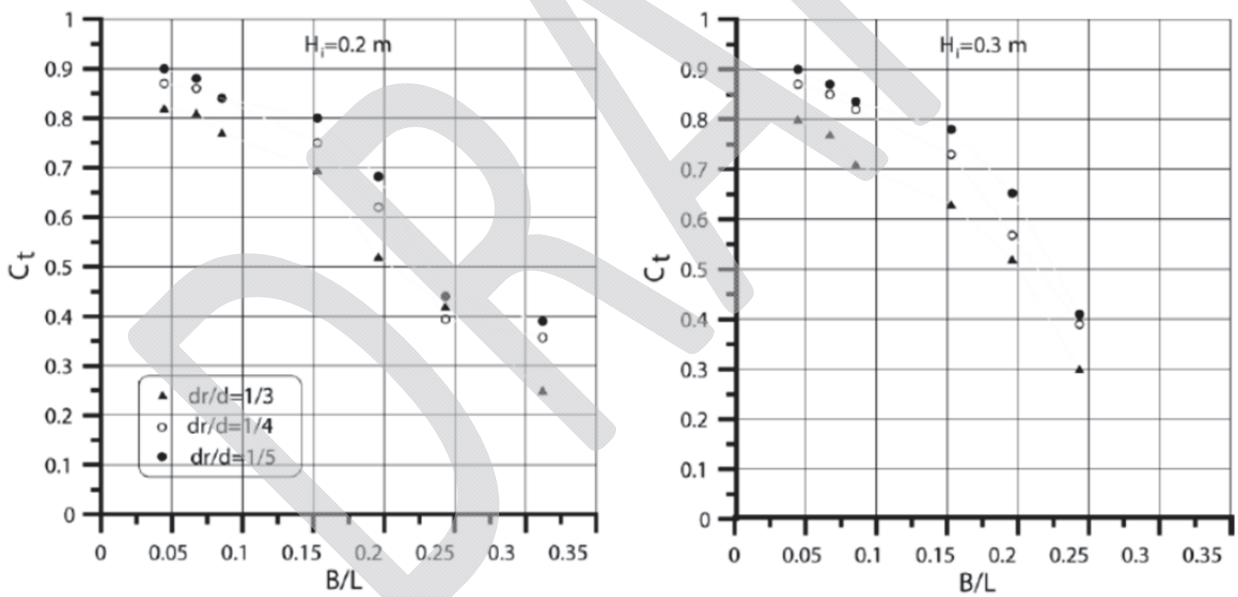

Figure D.3.1-5. Effect of Increasing Breadth on Transmission Coefficient for Regular Waves Koutandos (2005)

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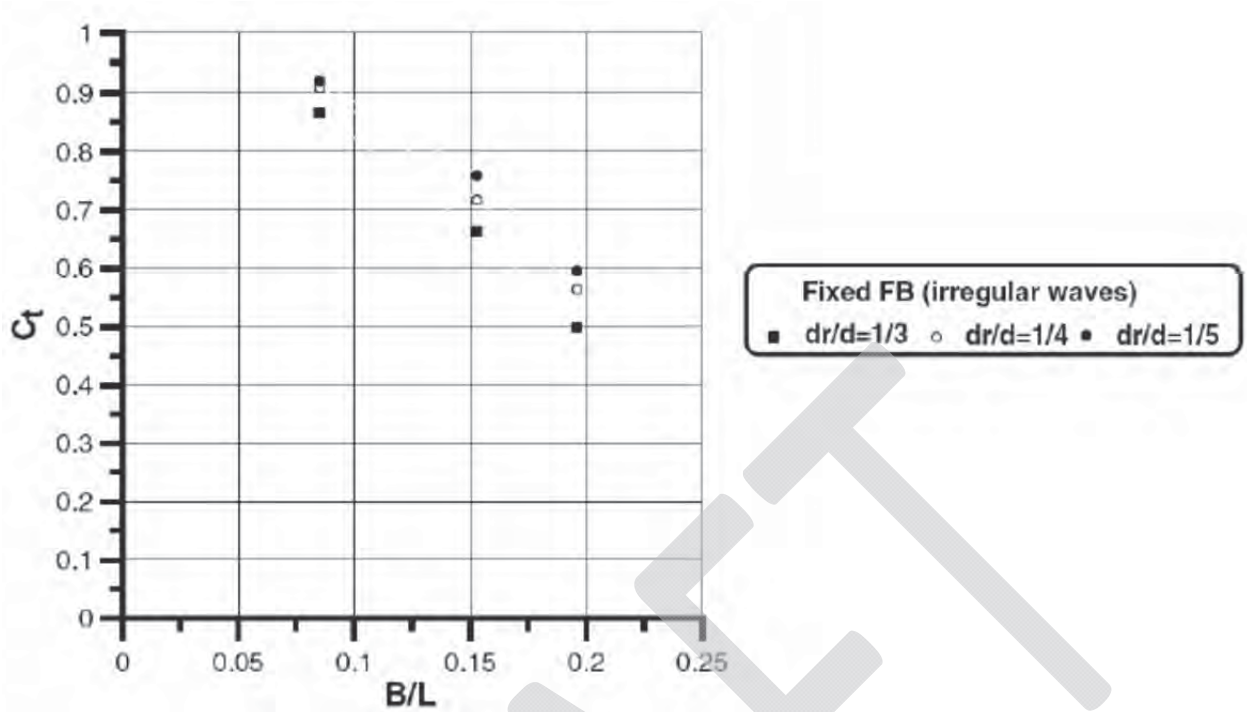

Figure D.3.1-6. Effect of Increasing Breadth on Transmission Coefficient for Irregular Waves Koutandos (2005)

D.3.1.1 Design Conditions

The incident wave height is an important design parameter that influences the design of the floating breakwater. The 50-year significant wave height of 6.9 feet based on the results of the Delft3D-wave numerical model simulations has been used for design. The corresponding wave period is 4.4 seconds. The required transmission coefficient is KT = 0.3, to meet the design criteria established in Section 2.1 (wave height inside the harbor not to exceed 2 feet in 50 years). The design parameters for the floating breakwater are summarized in Table D.3.1-1.

Table D.3.1-1. Floating Breakwater Design Conditions

Parameter	Value
Design wave Height (Hs, Feet)	6.9
Peak wave Period (Tp, s)	4.4
Water Depth (d, Feet)	30
Wavelength (L, Feet)	104
Transmission Coefficient (K_T)	0.3

D.3.1.2 Floating Breakwater Calculations and References

The experimental and the analytical methods discussed in the previous section have been analyzed for the design conditions at Portage Cove. Figure D.3.1-7 compares the experimental results of Tolba (1999) with the analytical methods. The data presented is for the D (draft)/d (depth) ratio of 1/6. The figure shows that when the breadth B increases, the transmission coefficient decrease for both the analytical and experimental methods. However, the

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analytical results predict higher values of transmission coefficients compared to experimental methods. The Carr, Healy and Stelzriede method matches the experimental results well for B/L values smaller than 0.2.

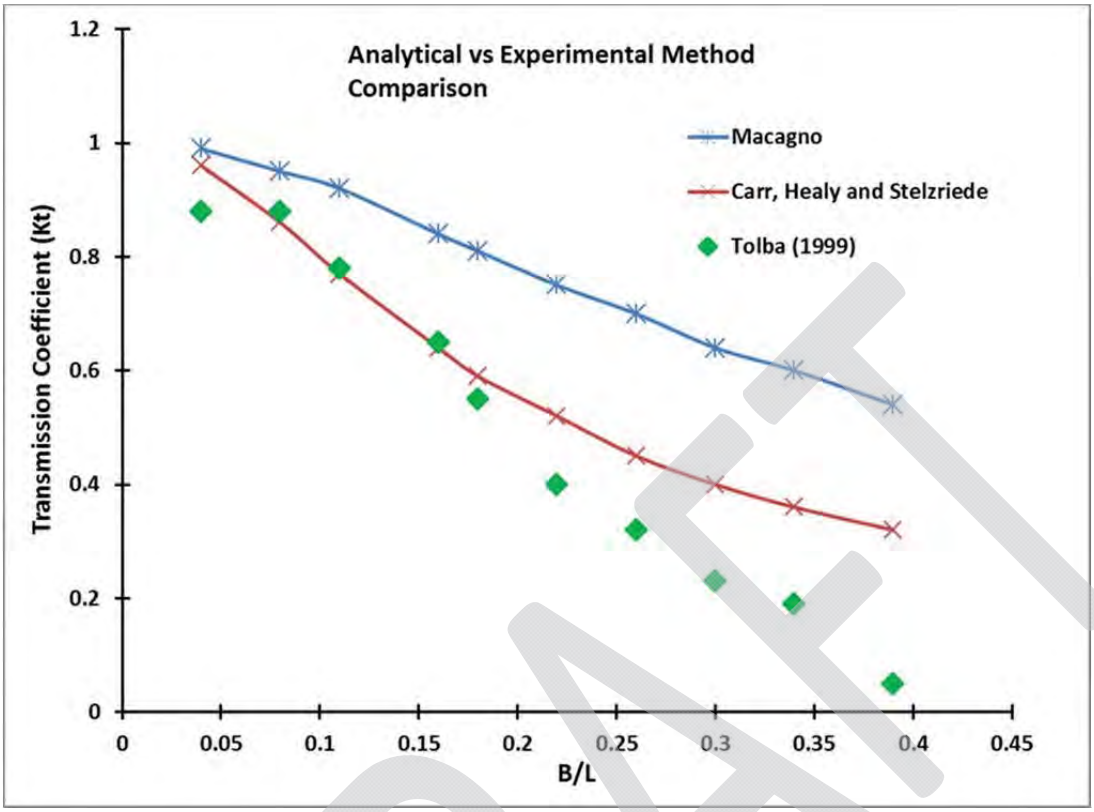

Figure D.3.1-7. Analytical and Experimental Method Comparison for Transmission Coefficients (Tolba)

The experimental results of Koutandos (2005), for different rations of D/d, have also been compared to the analytical methods, shown in Figure D.3.1-8. The Macagno values are slightly higher than the experimental results and the Carr, Healy and Stelzreide analytical method gives slightly lower values of transmission coefficients. For B/L value greater than 0.2 the Carr, Healy and Stelzreide method matches the experimental values.

Figure D.3.1-9 and Figure D.3.1-10 show the analytical expressions plotted for the design parameters depth (d) = 30 feet and wavelength (L) = 104 feet. The draft (D) is constant at 10 feet in Figure D.3.1-9. It can be seen that, with an increase in breadth the value of KT decreases more rapidly for the Carr, Healy and Stelzreide method. The plot shows that a breadth B= 40 feet is required for a transmission coefficient of 0.3, whereas a breadth B= 80 feet is required according to the Macagno expression to achieve the same transmission coefficient.

The breadth is kept constant at B=60 feet in Figure D.3.1-10. It is seen that, with increasing draft, the value of KT decreases rapidly according to the Carr, Healy and Stelzreide method. Note that for both methods the graph does not start at KT = 1.0, but at lower values. This indicates that a floating breakwater with a certain breadth already causes damping if the draft D = 0.

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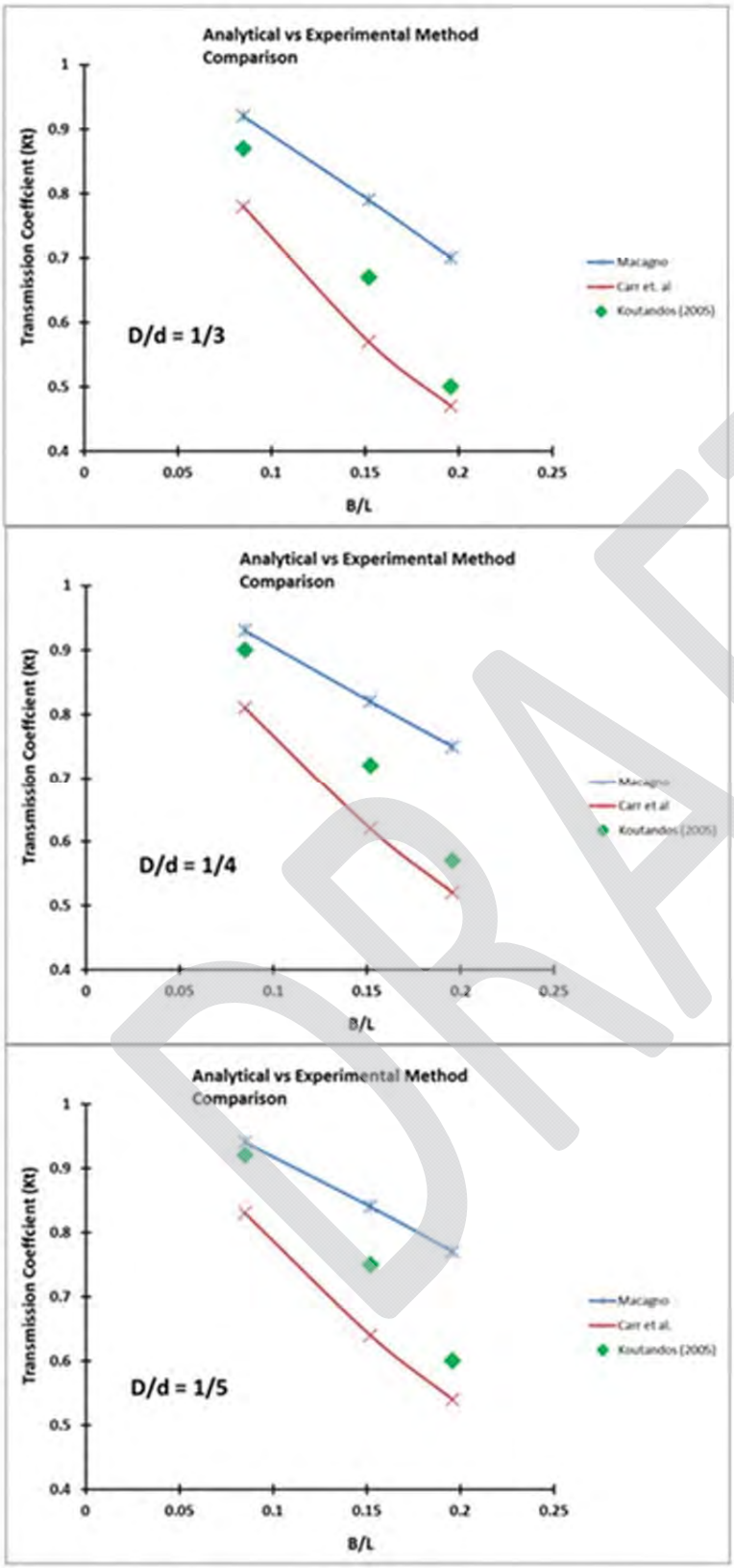

Figure D.3.1-8. Analytical and Experimental Method Comparison for Transmission Coefficients (Koutandos)

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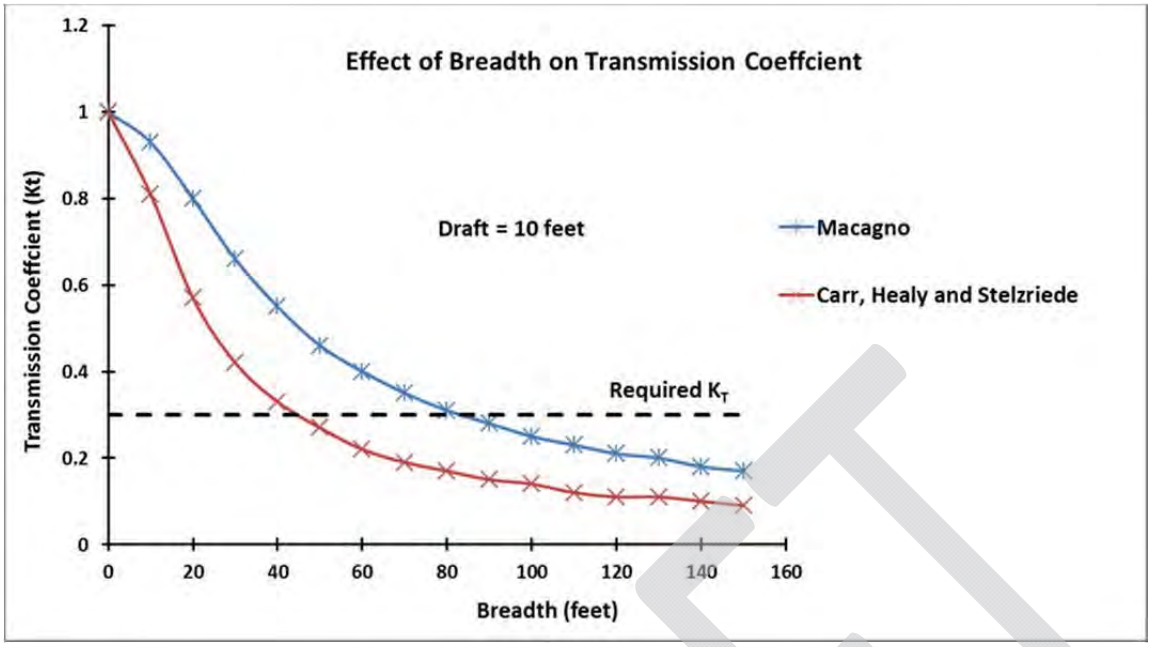

Figure D.3.1-9. Effect of Breadth on Transmission Coefficient

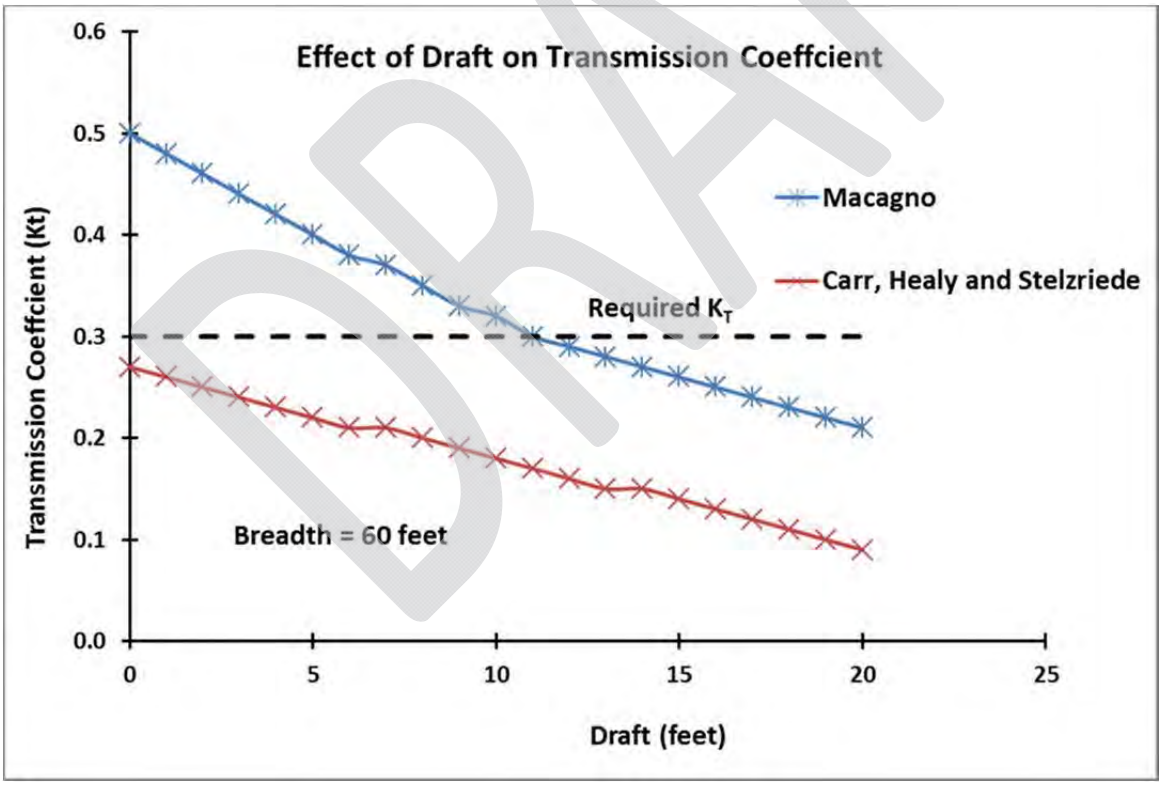

Figure D.3.1-10. Effect of Draft on Transmission Coefficient

The experimental and the analytical methods have been used to determine the breadth and draft of the floating breakwater to obtain the required transmission coefficient. Table D.3.1-2 summarizes the breadth and draft

combinations estimated from the different methods. Based on the results from the different methods a breadth B=50 feet and Draft D=10 feet will most likely yield the required transmission coefficient.

Table D.3.1-2. Dimensions of Floating Breakwater from Analytical and Experimental Methods

Reference	Breadth (B, feet)	Draft (D, feet)
Macagno	40	17
	50	13
	80	10
Carl, Healy and Stelzriede	30	12
	40	7
Tolba (1999)	26	10
Koutandos (2005)	27	10

D.4 Conclusions

The recommended Design Environmental Conditions (DEC) and Design Operational Conditions (DOC) for the Portage Cove Marine Facilities are summarized in Table 4.1-1 and Table 4.1-2. The DEC can be defined as the extreme conditions with a specific combination of tide, wind, waves and currents for which the facilities have to be designed. The DOC is defined as the limiting environmental condition that would require suspension of normal operations.

Table 4.1-1. Portage Cove – Design Operational Criteria (2-Yr Return Period)

Direction	Water Elevation (feet, MLLW)	Wind Speed (knots)	Wave	
			Hs (feet)	Tp (s)
Northeast (050°)	+17	31	2.6	2.5
East (090°)		31	2.1	2.2
Southeast (120°)		31	2.5	2.4

Table 4.1-2. Portage Cove – Design Environmental Criteria (50-Yr Return Period)

Direction	Water Elevation (feet, MLLW)	Wind Speed (knots)	Wave	
			Hs (feet)	Tp (s)
Northeast (050°)	+20	68	6.5	4.3
East (090°)		68	6.9	4.4
Southeast (120°)		68	6.3	4.3

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The following summarizes the key findings of the met-ocean analysis:

1. Prevailing winds are mostly from the east and northwest. However, large winds also occur from the northeast to southeast. The 50-year return period design wind speed is 68 knots.
2. The 50-year significant wave height is approximately 6.9 feet for winds from the east along a straight line fetch of 3 nautical miles. The waves from the northeast (6.5 feet) and southeast (6.3 feet) are slightly smaller since the waves experience diffraction and refraction effects before reaching the project site.
3. The estimated wind speeds and wave heights are consistent with the previous study done by USACE and local observation. No significant differences were noticed.
4. Tide range, defined as the distance between the Mean Higher High Water and Mean Lower Low Water, is 16.8 feet. However, the project site is located in an area which is experiencing glacial rebound and sea level fall. Hence it is reasonable to design for water levels approximately 2.5 feet below the mean higher high water.
5. Wave refraction causes large waves from Lynn Canal to approach the breakwater from a mostly easterly direction. Waves penetrate into the harbor through diffraction around the breakwater tip, and transmission under the breakwater (floating and partially penetrating vertical wall). The length and alignment of the breakwater alternatives is reasonable. The rubble mound breakwater has the most wave protection, and the floating breakwater the least, of the alternatives evaluated.

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D.5 References

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APPENDIX E

Wick Drain Analysis

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APPENDIX E:
Wick Drain Analysis

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E.1 Introduction

Wick drains (also known as prefabricated vertical drains) consist of a 4-inch by 1/8-inch wide geotextile wrap with a plastic insert. The plastic insert is composed of a variety of shapes and the primary purpose is to provide improved drainage from soil. They are installed vertically in soft, wet, compressible soils. The primary function of wick drains is to reduce length of the flow path thereby accelerating the removal of water from the soil. The accelerated removal of water from soil is analogous to squeezing water out of a sponge. As water is removed settlement occurs and strength gains occur in the soft soils as they compress. The primary advantages of using wick drains in offshore rubble-mound breakwater construction are:

- Reduced construction time,
- Accelerating settlement,
- Minimizing post construction settlement,
- Allowing strength gains to occur to aid in construction and improve post construction long term static and seismic stability.

PND's most recent experience installing wick drains offshore was for the USACE and Iraqi Navy in construction of a shoreside sheet pile dock system. The system was used to improve stability of the underlying Fao soft clay soils. The added benefit is accelerating settlement from in excess of 60 years to less than 3 months. Similar studies by PND of San Francisco airport expansion offshore resulted in decreasing settlement time from in excess of 250 years to less than 10 months.

The following sections describe the anticipated construction procedures for installing wick drains at the site and a discussion of the preliminary analysis of wick drain spacing and performance.

E.2 Typically Wick Drain Installation Procedure

Before installation can begin, the marine surface should be prepared as much of the proposed rubble-mound breakwater alignment is underlain with near-surface soft marine sediment. At this site, it will be necessary to place a sand or gravelly sand blanket a minimum of 3 feet thick to ensure proper performance of wick drains and to prevent the wick drains from settling or crimping in the soft marine sediment. This also promotes a constant flow of water out of the underlying soils as the load is applied during rubble-mound breakwater construction.

To maintain the desired spacing during wick drain installations, a template is typically constructed on the side of the barge. The position of the barge corners are located using a total station or GPS positioning. Once on station, using the barge anchoring system, the rig can insert the drains per the prefabricated layout pattern, typically in a triangular pattern spaced from 2.5 to 8 feet on center. Generally, three rows of drains can be installed for 5-foot-spacing from one barge position.

A specialized track-mounted excavator or crane is used to install wick drains (see Figure E.2-1). The wick drains come in rolls that are mounted near the base of the excavator/crane and threaded through a steel mandrel system that protects the wick drain from damage as it is installed through the soil. A sacrificial steel plate is attached to the wick drain at the base of the mandrel and used to minimize soil from entering the mandrel during the installation process. The installation force is typically provided by vibratory hammers, static force methods, or a combination of these methods depending on the soil conditions. Water may be used to lubricate the mandrel during installation to reduce the friction. Wick drains usually take less than a minute to install. The wick drain is cut several feet above ground, and a new sacrificial steel plate is fastened to the wick drain with staples. The plate is then retracted against the mandrel to minimize soil from entering the hollow mandrel. Once all the wick drains are installed, the construction of the rubble-mound breakwater may begin.

Figure E.2-1. Track-Mounted Excavator Installing Wick Drains

E.3 Preliminary Analysis of Wick Drain Spacing and Performance

The design of wick drains includes an evaluation of the site soil properties including determining the permeability and consolidation characteristics of the soils. The soil properties of the marine sediment were estimated using the results of laboratory testing completed as part of the geotechnical investigation (PND 2013) which included 7, one-dimensional consolidation tests. Based on the results of the marine sediment consolidation tests, the vertical coefficient of consolidation, c_v, which is the measure of the time rate of consolidation of the soil for a given load increment, ranged between about 75 and 205 ft^2/yr with an average value of 115 ft^2/yr.

Analysis of required wick drain spacing and performance was performed using the average c_v of 115 ft^2/yr based on our interpretation of the data and site soil conditions. A horizontal to vertical permeability ratio of 2:1 was assumed for the analysis. This ratio accounts for permeability in natural soil typically being greater in the horizontal than vertical direction and, generally, a range of 1:1 is appropriate for isotropic soils and 4:1 for highly layered (with sand seams) soil. The ratio of c_h to c_v back-calculated from full-scale tests was reported to range between 2 to 7 depending on the degree of stratification (Hansbo et al 1981).

Preliminary analysis of wick drain spacing and performance was evaluated using the Barron-Hansbo equation for vertical consolidation due to horizontal radial drainage (Barron, 1944, 1948; Hansbo, et al., 1981; Stamatopoulos and Kotzias, 1985). The equations predict the time of consolidation based on the material properties of the soil unit to be consolidated for a given wick drain configuration. It should be noted that this simplified analysis does not include the effects of either soil disturbance or drain resistance on the functionality of the wick drains. It may be appropriate to include these effects, and others, during final design of the wick drain spacing which is typically completed by the specialty contractor installing the drains and reviewed by the design team's geotechnical engineer.

Figure E.3-1presents estimates for percent consolidation versus time for various wick drain spacing for c_v equal to 115 ft^2/yr and Figure E.3-2 presents the results without use of wick drains for comparison.

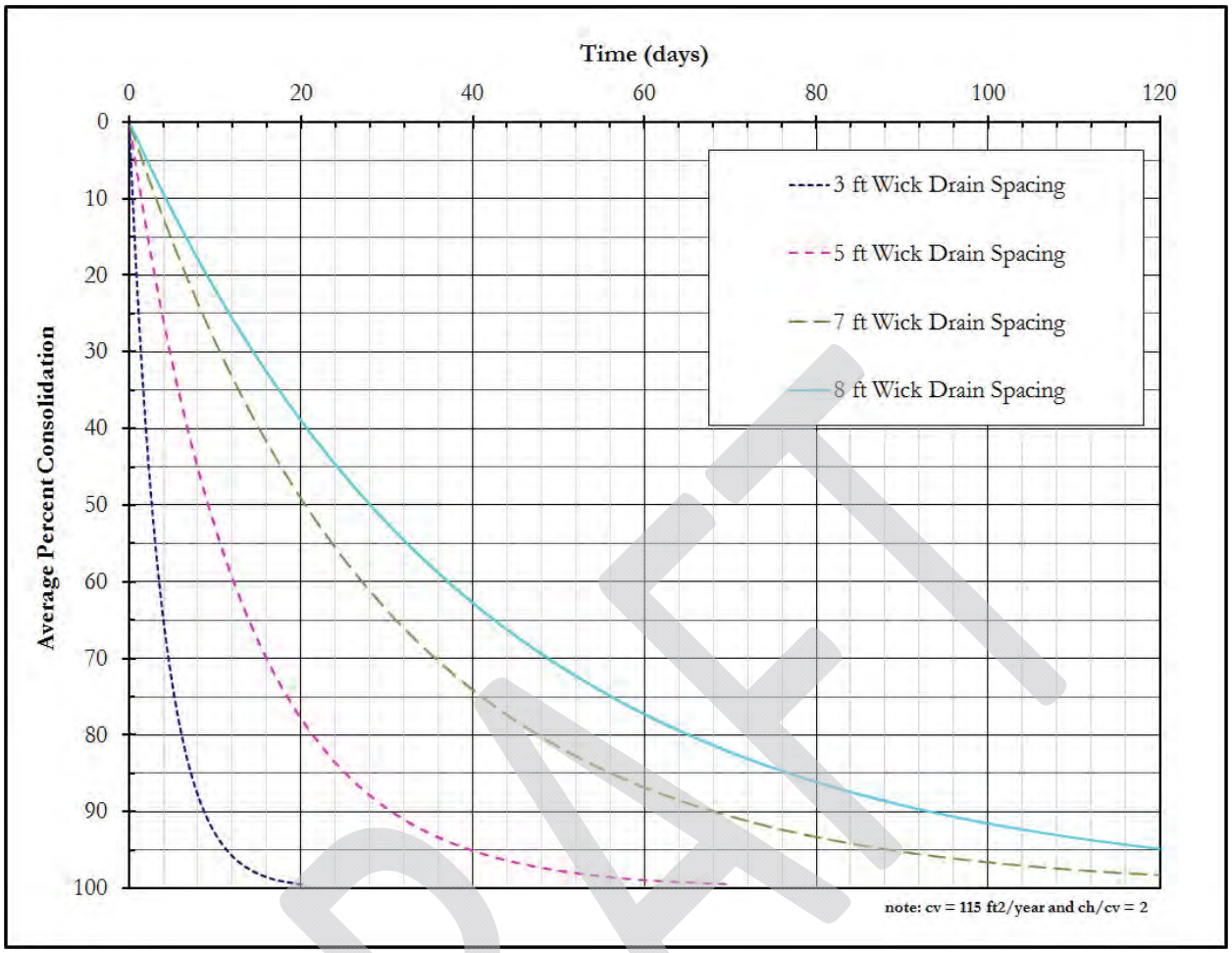

Figure E.3-1. Time Versus Average Degree of Consolidation for Various Triangular Wick Drain Spacing

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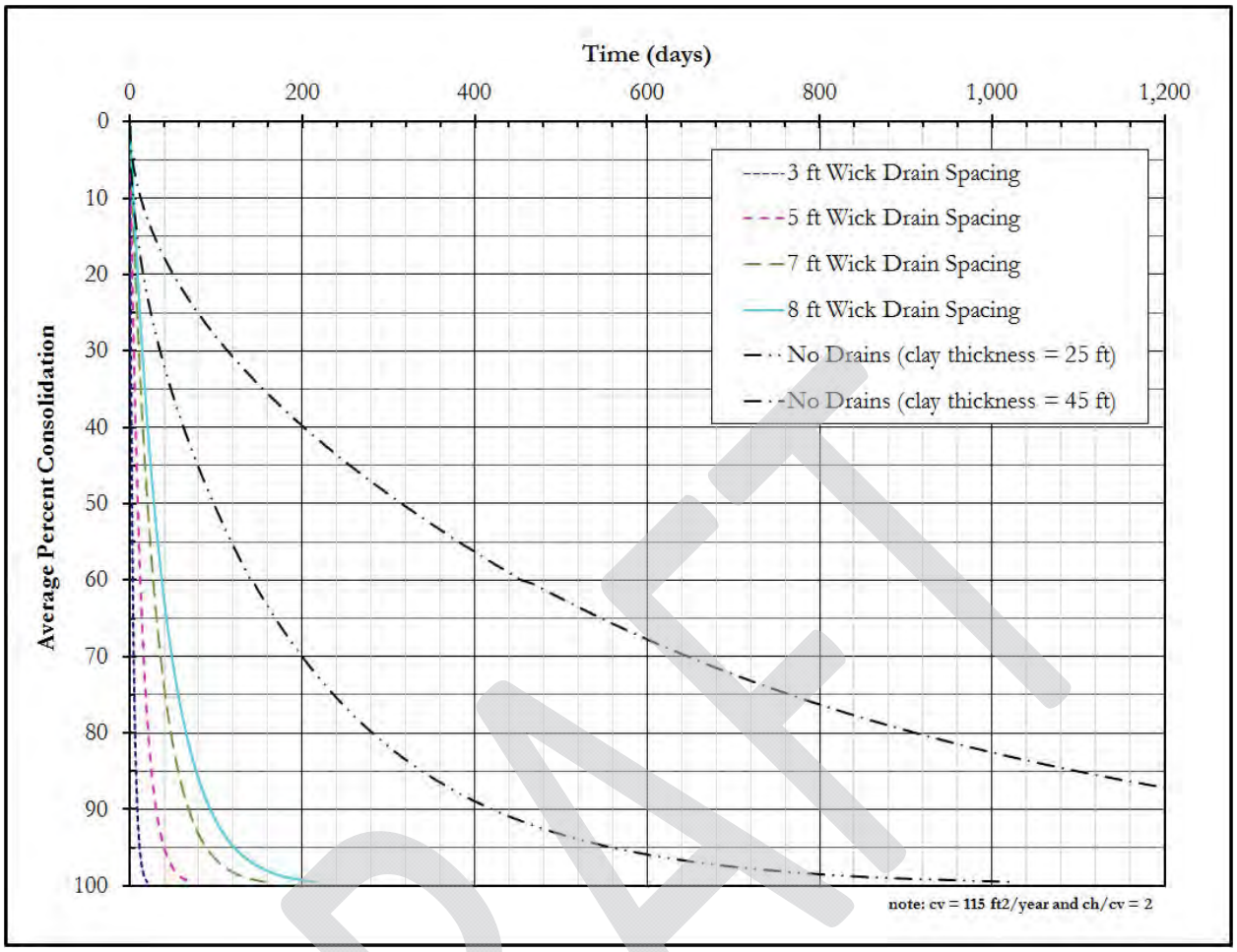

Figure E.3-2. Time Versus Average Degree of Consolidation for Various Triangular Wick Drain Spacing Without Wick Drains (c_v= 115 ft^2/yr).

The following table, Table E.3-1, summarizes the estimated time to reach 95% consolidation for an applied load with various wick drain spacing. The table also summarizes time to reach 95% consolidation without use of wick drains.

Table E.3-1. Estimated Time for 95% Consolidation for Various Wick Drain and No-Wick Drain Configurations

Wick Drain Configuration	Consolidation Time (95%) Days	Consolidation Time (95%) Months	Consolidation Time (95%) Years
3-ft Wick Drain Spacing	11	0.4	0.03
5-ft Wick Drain Spacing	40	1.3	0.11
7-ft Wick Drain Spacing	89	2.9	0.24
8-ft Wick Drain Spacing	121	4.0	0.33
No Drains (2-ft clay thickness)	560	18.4	1.53
No Drains (45-ft clay thickness)	1,814	59.6	4.97

Note: c_v=115 ft^2/yr; c_h/c_v=2

The results of this analysis indicate that excessively long time periods will be required to construct a rubble-mound breakwater without wick drains. Wick drains reduce construction time periods to reasonable values and are discussed in more detail in the main body of the report.

E.4 References

Barron, R. A. (1944). *The influence of drain wells on the consolidation of fine-grained soils*, Providence, R. I. District, U.S. Engineers Office.

Barron, R. A. (1948). "*Consolidation of Fine Grained Soils by Drain Wells.*" *Trans. ASCE*, 113, pp. 718-742.

Hansbo, S., M. Jamiolkowski, and L. Kok. (1981) "Consolidation by vertical drains." *Geotechnique*, 31, no. 1, pp. 45-66.

PND Engineers, Inc. (2013). *Haines Borough, South Portage Cove Harbor Expansion, Draft Final Geotechnical Report.*

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APPENDIX F

Cost Estimates

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SOUTH PORTAGE COVE HARBOR EXPANSION
RUBBLE-MOUND BREAKWATER
WITH WICK DRAINS
ALTERNATIVE 1
ENGINEER'S COST ESTIMATE

Item Description	Units	Quantity	Unit Cost	Item Cost
Wick Drain - Barge/Tug	Day	105	$10,000	$1,050,000
Wick Drain - Sand/Gravel Under Drain	CY	15,400	$40	$616,000
Wick Drain - Place Under Drain Material From Barge	CY	15,400	$15	$231,000
Wick Drain Installation (includes material)	LS	All Req'd	$1,565,000	$1,565,000
Prep Site for Placement	LS	All Req'd	$75,000	$75,000
Core Rock	CY	60,100	$35	$2,103,500
Armor Rock	CY	27,900	$65	$1,813,500
Underlayer Rock	CY	15,700	$50	$785,000
Place Material From Barge	CY	103,700	$15	$1,555,500
Construction Survey Measurement	LS	All Req'd	$150,000	$150,000
Navigational Aid	LS	All Req'd	$10,000	$10,000
Mobilization/Demobilization (8%)	LS	All Req'd	$796,360	$796,360
			Construction Subtotal	**$10,750,860**
			Construction Contingency (20%)	$2,150,172
			Environmental Permitting and Design (10%)	$1,075,086
			Construction Administration and Inspection (10%)	$1,075,086
			2-year Price Escalation (3%)	$916,618
			ESTIMATED TOTAL COST	**$15,967,822**

Assumptions:
Rubble Mound Material Costs Include Delivery and Placement
Wick Drain Costs per Estimate/Discussion with Hayward Baker (FOB Seattle)

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SOUTH PORTAGE COVE HARBOR EXPANSION
PARTIALLY-PENETRATING WAVE BARRIER, 550 feet

ALTERNATIVE 2
ENGINEER'S COST ESTIMATE

Item Description	Units	Quantity	Unit Cost	Item Cost
Furnish Fabricated Barrier Piles	LB	1,500,000	$1.20	$1,800,000
Install Barrier Piles	EA	105	$2,000	$210,000
Furnish Steel Barrier Cap	LB	140,800	$2.25	$316,800
Install Steel Barrier Cap	LF	550	$200	$110,000
Furnish Bearing Piles	LB	1,000,000	$1.20	$1,200,000
Install Bearing Piles	EA	38	$10,000	$380,000
Furnish Bearing Box Caps	LB	158,445	$3.00	$475,335
Install on Bearing Piles	EA	19	$6,000	$114,000
Connection to Barrier Cap	EA	19	$2,000	$38,000
Navigational Aid	LS	1	$10,000	$10,000
Misc. Amenities	LS	1	$150,000	$150,000
Mobilization/Demobilization (10%)	LS	All Req'd	$480,414	$480,414
			Construction Subtotal	**$5,284,549**
			Construction Contingency (20%)	$1,056,910
			Environmental Permitting and Design (10%)	$528,455
			Construction Administration and Inspection (10%)	$528,455
			2-year Price Escalation (3%)	$450,561
			ESTIMATED TOTAL COST	**$7,848,929**

Assumptions:
Bedrock is not encountered during pile installation
All Material Costs FOB Seattle - Freight Included in Mobilization/Demobilization
Galvanizing/Spray Metalizing Included in Unit Steel Prices Where Applicable
Cost Estimate Does Not Include Relocating Sewer Outfall, Dredging, or Other Harbor Improvements

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SOUTH PORTAGE COVE HARBOR EXPANSION
FLOATING WAVE ATTENUATOR, 500 feet x 50 feet
WITH CHAINS
ALTERNATIVE 3b
ENGINEER'S COST ESTIMATE

Item Description	Units	Quantity	Unit Cost	Item Cost
Concrete Floating Wave Attenuator	CF	250,000	$25	$6,250,000
Anchor Chains (90' shots) - galvanized carbon steel	EA	135	$1,900	$256,500
Install Chains	EA	12	$10,000	$120,000
Shackles	EA	150	$36	$5,400
Furnish anchor piles (60' long, 24" dia) with spin fins	LB	90,439	$2.00	$180,878
Install anchor piles	EA	12	$15,000	$180,000
Miscellaneous	LS	All Req'd	$25,000	$25,000
Navigational Aid	LS	All Req'd	$10,000	$10,000
Mobilization/Demobilization (10%)	LS	All Req'd	$702,778	$702,778
			Construction Subtotal	**$7,730,556**
			Construction Contingency (20%)	$1,546,111
			Environmental Permitting and Design (10%)	$773,056
			Construction Administration and Inspection (10%)	$773,056
			2-year Price Escalation (3%)	$659,107
			ESTIMATED TOTAL COST	**$11,481,886**

Assumptions:
Bedrock is not encountered during pile installation
All Material Costs FOB Seattle - Freight Included in Mobilization/Demobilization
Galvanizing/Spray Metalizing Included in Unit Steel Prices Where Applicable

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SOUTH PORTAGE COVE HARBOR EXPANSION
FLOATING WAVE ATTENUATOR, 500 feet x 50 feet
WITH PILES
ALTERNATIVE 3a
ENGINEER'S COST ESTIMATE

Item Description	Units	Quantity	Unit Cost	Item Cost
Concrete Floating Wave Attenuator	CF	250,000	$25	$6,250,000
Pile Hoops - Furnish and Install	EA	12	$10,000	$120,000
Furnish Pile	LB	422,050	$1.20	$506,460
Install Piles	EA	24	$15,000	$360,000
Miscellaneous	LS	All Req'd	$25,000	$25,000
Navigational Aid	LS	All Req'd	$10,000	$10,000
Mobilization/Demobilization (10%)	LS	All Req'd	$727,146	$727,146
			Construction Subtotal	**$7,998,605**
			Construction Contingency (20%)	$1,599,721
			Environmental Permitting and Design (10%)	$799,861
			Construction Administration and Inspection (10%)	$799,861
			2-year Price Escalation (3%)	$681,961
			ESTIMATED TOTAL COST	**$11,880,009**

Assumptions:
Bedrock is not encountered during pile installation
All Material Costs FOB Seattle - Freight Included in Mobilization/Demobilization
Galvanizing/Spray Metalizing Included in Unit Steel Prices Where Applicable
Cost Estimate Does Not Include Relocating Sewer Outfall, Dredging, or Other Harbor Improvements

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SOUTH PORTAGE COVE HARBOR EXPANSION
LIFE CYCLE COST ESTIMATES
ENGINEER'S COST ESTIMATE

Note: all values in 2013 dollars (net present value)

Rubble-Mound Breakwater (Alternative 1)

Item Description	Units	Freq.	Quantity	Unit Cost	Item Cost	Remarks
Operation and Maintenance	YR	1	1	$500,000	$500,000	25-year cycle for armor/scour repair
Inspection	LS	11	1	$5,000	$55,000	5-year cycle and after major storm (assume 2)
Replace Navigational Aid	EA	4	1	$10,000	$40,000	10-year cycle
				Total	**$ 595,000**	

Partially-Penetrating Wave Barrier (Alternative 2)

Item Description	Units	Freq.	Quantity	Unit Cost	Item Cost	Remarks
Inspection	LS	9	1	$30,000	$270,000	5-year cycle
Anode Furnish Install	EA	7	90	$1,000	$630,000	7-year cycle: 90, 80 lb. anodes
Replace Navigational Aid	EA	4	1	$10,000	$40,000	10-year cycle
				Total	**$ 940,000**	

Floating Wave Attenuator (Alternative 3b)

Item Description	Units	Freq.	Quantity	Unit Cost	Item Cost	Remarks
Inspection	LS	12	1	$10,000	$120,000	2-year cycle until year 25
Inspection	LS	25	1	$10,000	$250,000	1-year cycle after year 25
Concrete Crack Repair	LS	4	1	$75,000	$300,000	10-year cycle
Anode Furnish Install	EA	7	24	$1,000	$168,000	7-year cycle: 24, 80 lb. anodes
Replace Pile Hoops	EA	3	12	$15,000	$540,000	15-year cycle
Replace Navigational Aid	EA	4	1	$10,000	$40,000	10-year cycle
				Total	**$ 1,418,000**	

Floating Wave Attenuator (Alternative 3a)

Item Description	Units	Freq.	Quantity	Unit Cost	Item Cost	Remarks
Inspection	LS	12	1	$10,000	$120,000	2-year cycle until year 25
Inspection	LS	25	1	$10,000	$250,000	1-year cycle after year 25
Concrete Crack Repair	LS	4	1	$75,000	$300,000	10-year cycle
Furnish Upper Anchor Chain Shot	EA	9	12	$2,000	$216,000	5-year cycle: 12 Anchor chains
Install Upper Anchor Chain Shot	EA	9	12	$1,500	$162,000	5-year cycle: 12 Anchor chains
Shackles for Anchor Chain Shot	EA	9	24	$36	$7,776	5-year cycle: 12 Anchor chains
Replace Navigational Aid	EA	4	1	$10,000	$40,000	10-year cycle
				Total	**$ 1,095,776**	

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