CITY OF PORT LAVACA

MEETING: FEBRUARY 18, 2025 **AGENDA ITEM ____**

DATE: 2.18.2025

TO: PORT COMMISSION BOARD MEMBERS CC: JIM RUDELLAT, HARBOR MASTER

FROM: JODY WEAVER, INTERIM CITY MANAGER

SUBJECT: **CITY MANAGER'S MONTHLY REPORT**

- TPWL Grant – Renovations to the Nautical Landings Marina Breakwater:
  At the City Council meeting on February 10, Council awarded a construction contract to Derrick Construction for $445,162.00. We are working on getting the contract signed and scheduling a pre-construction conference. Would the Commission like to select a member to attend the pre-construction conference as a representative of the Port Commission?

- CDBG-MIT Coastal Resilience Living Shoreline Project
  No new update information.

- ReStore (cleanup of old barge(s) in Smith Harbor)
  Our consultant Kim Griffith reported last week that the National Wide Permit application was ready for submittal except she was waiting on one last piece of information from the COE before having us sign and finally submit. This delay will translate into a delay in receiving authorization to bid as well.

- CDBG-MIT Round 2 Application for use of funds for Replacement of culverts under rail at Corporation Ditch and Voluntary Restoration of Refuge Shoreline. No new information.

- GLO CEPRA GRANT (Harbor of Refuge Shoreline Protection): Attached is the final Alternative Analysis Report. Alternative 4A has been selected (see pg. 30). This is a breakwater configuration along the harbor peninsula and southern shoreline consisting of 6 shingles segmented rubble mounds breakwaters providing fish gaps with an extended gap protection overlap. The shingle orientation is to protect against the largest waves which come from the SE direction.

- MBMT Grant Downtown Waterfront Public Access Improvement:
  We have received the Nationwide permit for the bulkhead but are waiting on the permit for the docks.

- TxDOT Truck Route signs: I reached out to TxDOT for an update yesterday and received an email back that a pre-construction conference had not been scheduled yet and it was copied to the TxDOT PM asking him to provide an update for us.

City of Port Lavaca – Harbor of Refuge Project

Coastal Engineering Analysis and Alternatives Analysis

Project:	City of Port Lavaca – Harbor of Refuge Project		
Prepared by:	C. Johnson, A. Hnatow, T. Everett	**Date:**	1/24/2024
Approved by:	J. Carter	**Checked by:**	T. Everett
Subject:	Coastal Engineering Analysis and Alternatives Analysis		

1 Introduction

A coastal engineering and alternatives analysis was conducted by Mott MacDonald (MM) and Coast and Harbor Engineering (CHE) to develop shoreline protection measures along the Harbor of Refuge shoreline. The analysis was performed to increase project site understanding, aid in numerical modeling, evaluate alternatives, and develop a shoreline protection system. The purpose of the shoreline protection system is to reduce wave energy during strong cold fronts and tropical cyclones (TC) while maintaining existing aquatic habitat. Note that the breakwater systems investigated are not designed to reducing tidal or storm driven water levels along the project shoreline.

The goal of the data collection and modeling efforts were to select a preferred alternative. The data collection analysis characterized daily environmental conditions and wave climate at the project site. Various survey data were compiled to create a continuous bathymetry model for the project vicinity that was then used as input for modeling efforts. Numerical models, such as SWAN and XBeach, were used to generate boundary conditions, simulate design conditions, and evaluate shore protection alignments. Wave transmission behind the breakwater system was used as the key performance criterion to evaluate the alternatives.

2 Data Collection and Analysis

Wind, water level, sea level rise, tidal datum, and wave data near the project site were collected from a variety of sources to characterize extremal and day-to-day conditions at the project site. The results of this analysis will be used as input for the numerical modeling to aid in alternative design and are summarized in the following sections.

2.1 Datums

Based on its proximity to the project site, the tidal datums for the project site were acquired from the National Oceanic and Atmospheric Administration (NOAA) Station No. 8773259 at Port Lavaca, TX. Tidal datums for the tidal epoch of 2002-2006, relative to the North American Vertical Datum of 1988 (NAVD88) were collected via the NOAA Tides and Currents website and are presented in Table 2.1. All elevations reported within this memorandum are referenced to NAVD88, unless otherwise specified.

Table 2.1: Tidal Datums at the Port Lavaca Station referenced to NAVD88.

Datum	Elevation [ft NAVD88]
Mean Higher High Water (MHHW)	1.2
Mean Sea Level (MSL)	0.79
Mean Lower-Low Water (MLLW)	0.3
North American Vertical Datum of 1988 (NAVD88)	0

2.2 Wind, Water Level, and Wave Data

Wind, water level, and wave data were collected from a variety of sources. Historical wind and water level data were collected from nearby NOAA stations as shown in Table 2.1. The low frequency of occurrence (extremal) water levels and waves were calculated using data from the NOAA stations and other sources as described in Section 2.3.

Table 2.2: Summary of historical gauge and probabilistic data collected.

Source	Data Type	Data Range
Port Lavaca, TX NOAA 8773259	Water level Wind	2015 – 2024 2013 – 2024
Port O'Connor, TX NOAA 8773701	Water level Wind	2015 – 2024 2013 – 2024

2.2.1 Water Level Data & Relative Sea Level Rise

Water level data from the Port Lavaca NOAA station was analyzed to characterize day-to-day conditions. The Port Lavaca NOAA Gauge 8773259 shows a tidal range of approximately 0.81 feet from MHHW to MLLW. Extremal conditions were developed with the Port Lavaca NOAA station's data using the methodology described in Section 2.3.

In addition to the water level analysis, relative sea level rise (RSLR) predictions were calculated for the project site. RSLR reflects changes in local mean sea level (LMSL) over time and is a combination of eustatic sea level rise and local land movement, e.g., subsidence. The Special Report of the Intergovernmental Panel on Climate Change (SROCC) projects future eustatic Sea Level Rise (SLR) rates up to the year of 2100 and include an intermediate (RCP4.5) and a high (RCP8.5) emission scenario, the latter leading to more severe sea-level rise than the former.

Vertical Land Movement (VLM) is monitored at Rockport, TX and Freeport, TX by means of a Continuously Operating GPS Reference Station (CORS). The VLM rate of Rockport and Freeport is -2.88 mm/yr and -2.45 mm/yr, respectively. The CORS VLM rates generally agree with those estimated from collocated, long-term NOAA tide gauges: -5.97 mm/yr and -3.66 mm/yr, respectively. The CORS VLM rates are direct observations and are preferred to the NOAA estimations. The VLM rate observed at Rockport was selected as the conservative choice. Further, the VLM rate is assumed to be constant throughout the project lifetime.

Using the SLR and VLM data discussed above, the future RSLR projections were calculated to the year 2100. The projections are shown in Figure 2.1.

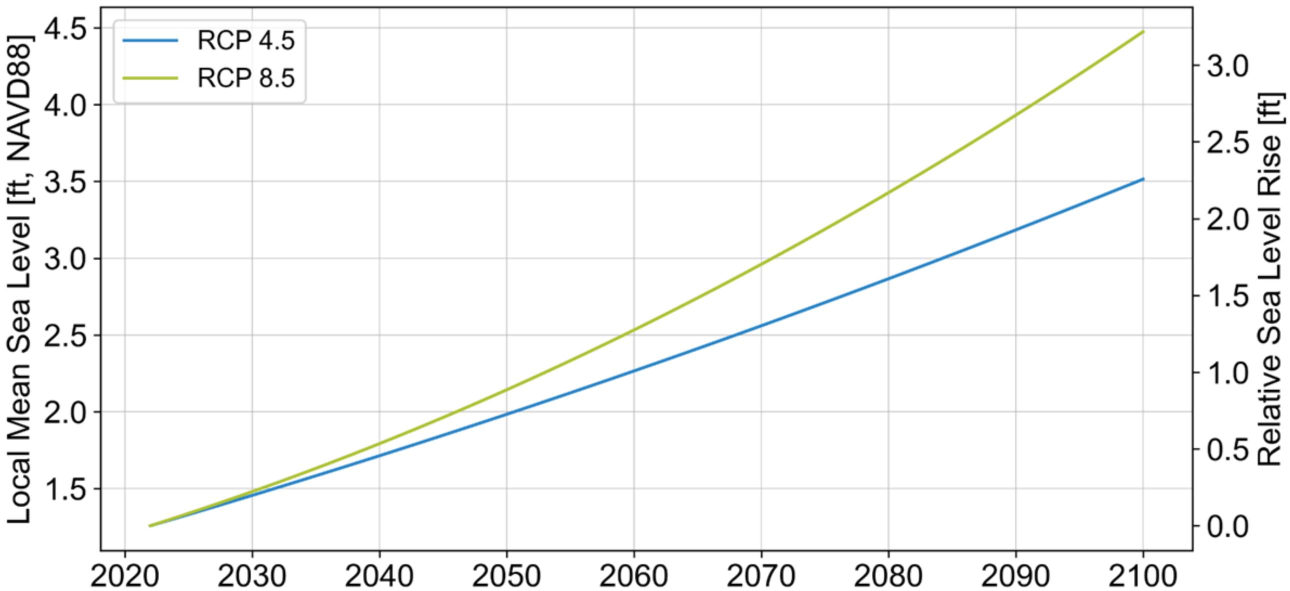

Figure 2.1: SROCC RSLR Projections for Port Lavaca, TX.

Assuming RSLR at the intermediate rate, water levels at the project site will be approximately 0.3 ft higher in 10 years and 1.4 ft higher in 50 years. Assuming the RSLR at the high rate, water levels at the project site will be approximately 0.3 ft higher in 10 years and 1.8 ft higher in 50 years. The intermediate RSLR projections will be taken into consideration for design of any alternatives to ensure the longevity of the design over the life of the project. See Table 2.3 for projected values of RSLR and LMSL at the project site.

Table 2.3: Projected relative sea level rise and local mean sea level at Port Lavaca.

Year	RSLR [RCP 4.5] (ft)	LMSL [RCP 4.5] (ft, NAVD88)	RSLR [RCP 8.5] (ft)	LMSL [RCP 8.5] (ft, NAVD88)
2024	0.00	1.3	0.00	1.3
2034	0.3	1.5	0.3	1.5
2049	0.6	1.9	0.8	2.1
2074	1.4	2.6	1.8	3.1

2.2.2 Wind Data

Wind data were collected from the Port Lavaca and Port O’Connor NOAA stations. The Port Lavaca station is the closest to the project site and was therefore used to quantify typical wind patterns. A wind rose for this station is shown in Figure 2.2, which illustrates the relative frequency of observed winds within 16 directional bins and 5 magnitude bins separated by month. As shown in the wind rose, stronger winds at the project site are predominantly from the south to southeast, typically associated with tropical storm events that occur during the summer months.

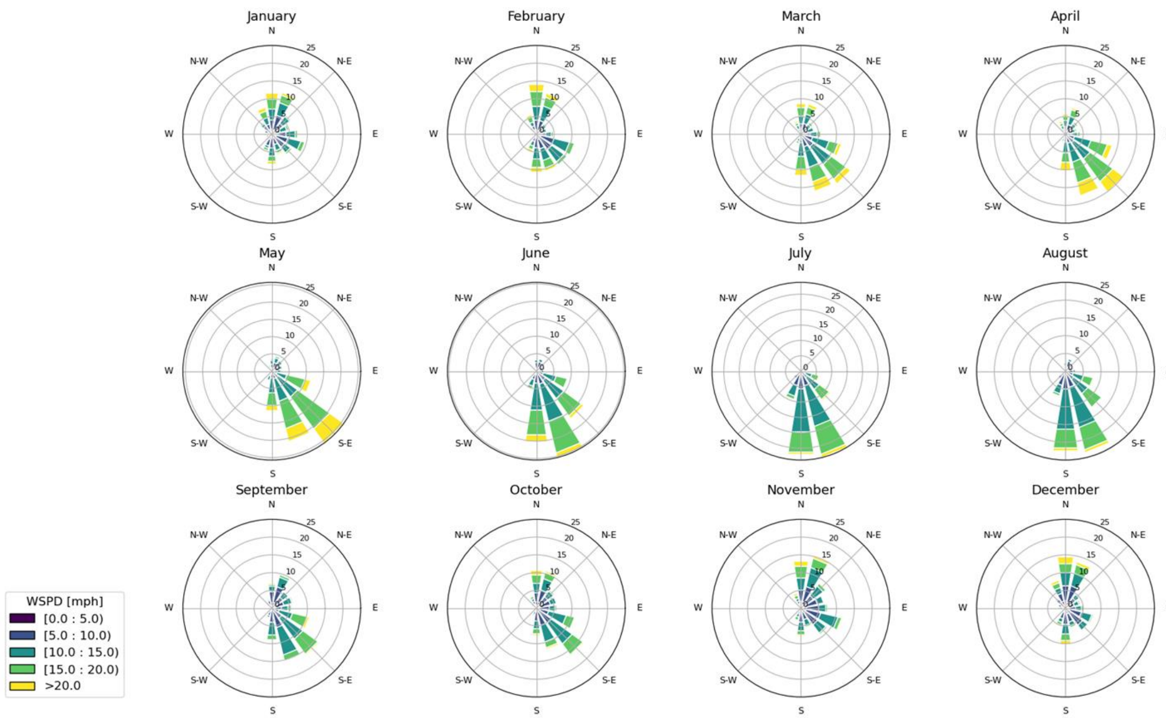

Figure 2.2: Seasonal wind rose at NOAA Station 8773259.

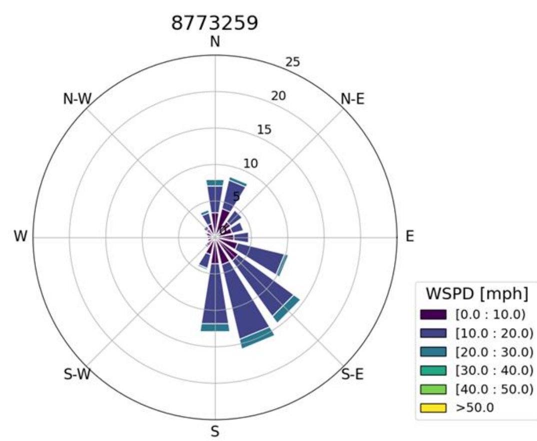

Figure 2.3: Overall wind rose at NOAA Station 8773259.

2.2.3 Wave Climate

The wave climate for Lavaca Bay was simulated using the SWAN (Simulating WAves Nearshore) wave model driven by the wind and water level data from nearby the NOAA gauge. The details of the long-term wave model can be found in Section 3.1.1.

Winds and water levels from 2016 to 2024 were acquired from the Port Lavaca NOAA gauge. The winds were corrected to 10 meters above the sea surface and applied uniformly to the model. The water levels were also imposed uniformly across the model domain. Wave parameters were extracted offshore of the project shoreline at approximately -2.25 ft NAVD88 (see Figure 3.1 for the extraction location). The time series of the wind and water level inputs and simulated wave parameters are presented in Figure 2.4.

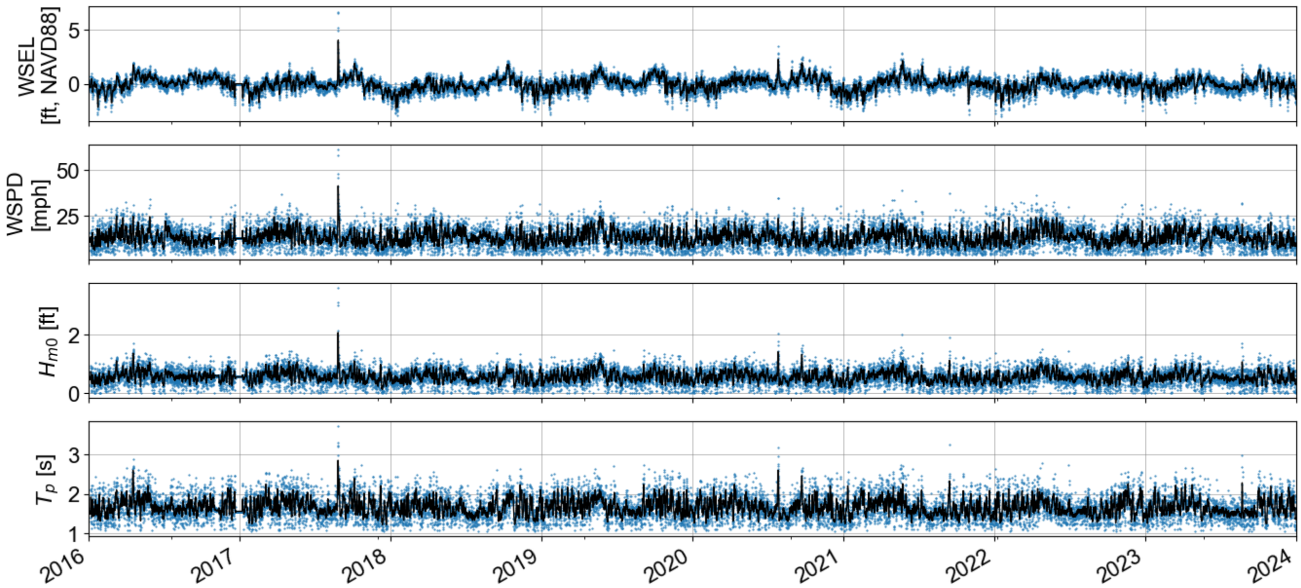

Figure 2.4: Wave hindcast results used to determine the wave climate. WSEL is water surface elevation, WSP is wind speed, H_{m0} is the zero-moment (significant) wave height, and T_p is the peak wave period. The blue dots indicate individual data points, and the black line is a running average provided for visual clarity. The WSP and WSEL data were observed at the Port Lavaca NOAA gauge and wave parameters were extracted from the long-term wave model offshore of the site (see Figure 3.1).

To characterize the wave climate, the time series wave parameters were analyzed and converted into a wave rose. A wave rose, similar to a wind rose, shows the percentage of occurrence of a given wave state in terms of magnitude and direction. Figure 2.5 shows the wave roses (wave height and wave period) for the project site.

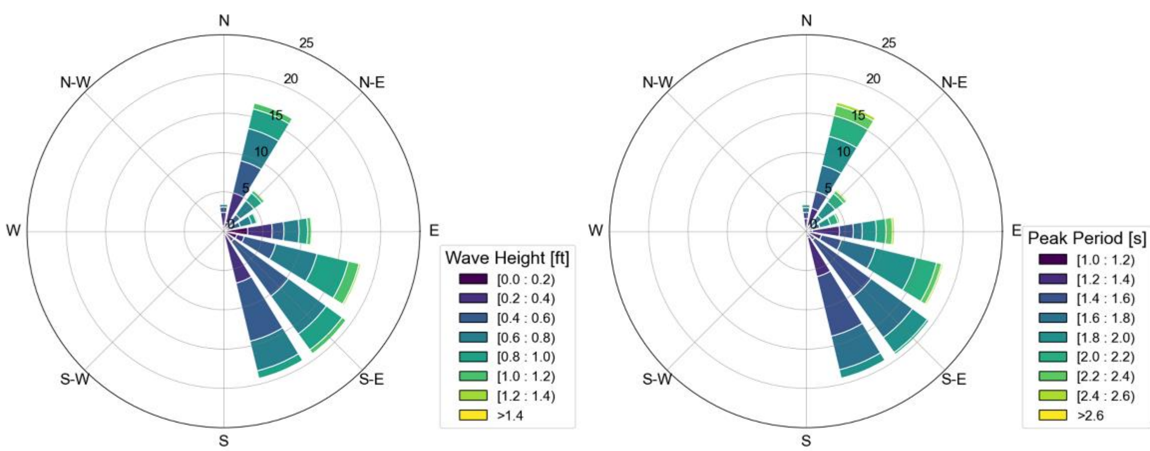

Figure 2.5: Wave roses for project site with wave height and peak period on the left and right, respectively.

The wave roses show two sea-states which characterize the wave climate at the project site. The first is southeasterly waves which are generated by winds out of the southeast blowing across Matagorda and Lavaca Bay. The strongest waves in this mode are from the east-southeast. The second sea-state is comprised of northeasterly waves, presumably generated by the passage of cold fronts driving northerly winds across Lavaca Bay.

Due to the limited northern fetch at the project site (approximately 8.5 miles), the cold-front generated sea-state is less energetic and comprises a smaller percentage of the wave rose than is observed in other parts of

Matagorda Bay. Also, due to long south-easterly fetches (the site is exposed to 25 miles of open Matagorda Bay fetch from the ESE), the first sea-state is more energetic and forms a larger percentage of the typical seas.

2.3 Extremal Analysis

An extremal analysis was conducted to develop extremal water surface elevation, wind, and waves at the project site. The extremal analysis used the Yearly Maximum and Peaks Over Threshold Methods. An extremal analysis of winds and water levels used multiple data sources to obtain low frequency return period events, i.e., return period (T_r) greater than 10 years. The extremal analysis of the wave conditions builds on the water level and wind analysis by utilizing those return period conditions to drive simulated wave conditions at the site. The wind and water level and wave extremal analyses are described in Section 2.3.1 and Section 2.3.2, respectively.

2.3.1 Wind and Water Level

The extremal analysis of the Port Lavaca NOAA gauge data excludes TCs due to the insufficient length of the gauge's observation record. Observational record length determines the accuracy and confidence associated with estimating low-probability events such as hurricanes and tropical storms. To characterize extreme events, the gauge data analysis was supplemented with additional data sources. The average of extremal water level conditions across the project location is summarized in Table 2.4. The extremal wind conditions are also shown in Table 2.4.

TC track data from the National Hurricane Center's (NHC) International Best Track Archive for Climate Stewardship (IBTrACS) database (NOAA, 1987) were analyzed to provide extremal wind speed values for $T_r > 10$ years. The IBTrACS database spans from 1842 to 2015 and contains storm track, wind speed, and pressure data. Wind speeds were extracted for all storms passing within 75 nautical miles of the project site. Wind speeds from the IBTrACS database were used in the extremal analysis and the results are summarized in Table 2.4.

To supplement the extremal water level analysis, the Sabine to Galveston (S2G) storm-induced coastal flooding study was utilized ($T_r \geq 10$ years). The S2G study was conducted by the United States Army Corps of Engineers (USACE) as part of their Coastal Hazards System (CHS). The S2G study simulated coastal flooding generated by an exhaustive set of TC tracks, enhancing the set of known historical storms with synthetic TC tracks of known likelihood. The enhanced dataset allows for inferring low-probably storm-induced water levels along the coast. Extremal water levels with return periods between 10 and 100 years in Table 2.4 were extracted from the S2G study in close proximity to the project area (save point 1274).

Table 2.4: Summary of extremal statistics at project site. Note that the extremal statistics are representative of normal conditions at the project site. NOAA gauge data do not include the effects of tropical cyclones.

Return Period [year]	Water Level (non-TC) [ft NAVD88] [1]	Water Level (TC) [ft NAVD88] [2]	Wind Speed [mph] [3]
1	2.7	-	25 (NOAA)
5	3.7	-	33 (NOAA)
10	4.0	2.0	37 (NOAA)
25	4.4	4.9	79 (NHC)
50	4.8	7.4	92 (NHC)
100	5.2	8.9	105 (NHC)

[1] Hurricane Bill (2005), Hurricane Harvey (2017), and Hurricane Hannah (2020) were removed from the extremal water level analysis.
[2] S2G study was used to determine water levels for tropical cyclones for $T_r > 10$ yr.
[3] Extremal wind speeds from NOAA gauge data ($T_r = 1 - 10$ yr) and NHC data ($T_r > 10$yr).

2.3.2 Waves

An extremal analysis on the wave hindcast is of limited value due to the length of the record. Therefore, an alternative approach was followed. The results of the extremal wind and water level analysis were used to model wave generation at the project site. Since the project site is within an enclosed coastal embayment, the sea-state will be governed by local winds for a given water level. While water level and local wind are not as correlated as local winds and sea-state, the probability of a given extremal wind and water level (e.g., a 20-year return period) occurring simultaneously is lower than an independent occurrence. Therefore, this assumption will lead to conservative design values.

The extremal analysis of the wind does not consider wind direction, but rather wind speed alone. Therefore, a set of wind directions corresponding to the maximum fetches at the project site were modelled for each return period condition.

A separate wave model was created for the extremal wave modeling which used finer resolution at the project site and within Lavaca Bay (see Section 3.1.2 for model details). This was done to achieve sufficient resolution at the project site for additional detailed modeling of the alternatives. Since the extremal wave modeling and alternative analysis required only modeling tens of wave conditions, it was computationally feasible. However, modeling waves at this resolution for the hindcast, which simulated tens of thousands of wave conditions, was not computationally feasible and was not necessary for characterizing the project site's wave climate. The results of the extremal wave modeling are presented within alternatives analysis section (Section 5.2) in Figure 5.4 through Figure 5.7 for return periods of 1-, 5-, 25-, and 100-year, respectively.

2.4 Habitat Survey

A habitat survey of the project site and vicinity was conducted between June 24 – 25 and July 17 – 25, 2024 by Triton Environmental. The habitat survey boundaries are shown in Figure 2.6 which includes data from National Wetland Inventory (NWI). The overall habitat survey results are shown in Figure 2.7. A total of 87 transects and 3,420 sample stations were taken. The results found a total of 9.12 acres of live oyster reef. No seagrass was encountered. The sensitive habitat found is shown in Figure 2.8. The survey identify a live oyster bed near the southern end of the proposed breakwater. The breakwater alignment will include the appropriate buffer to avoid any impacts to this oyster bed.

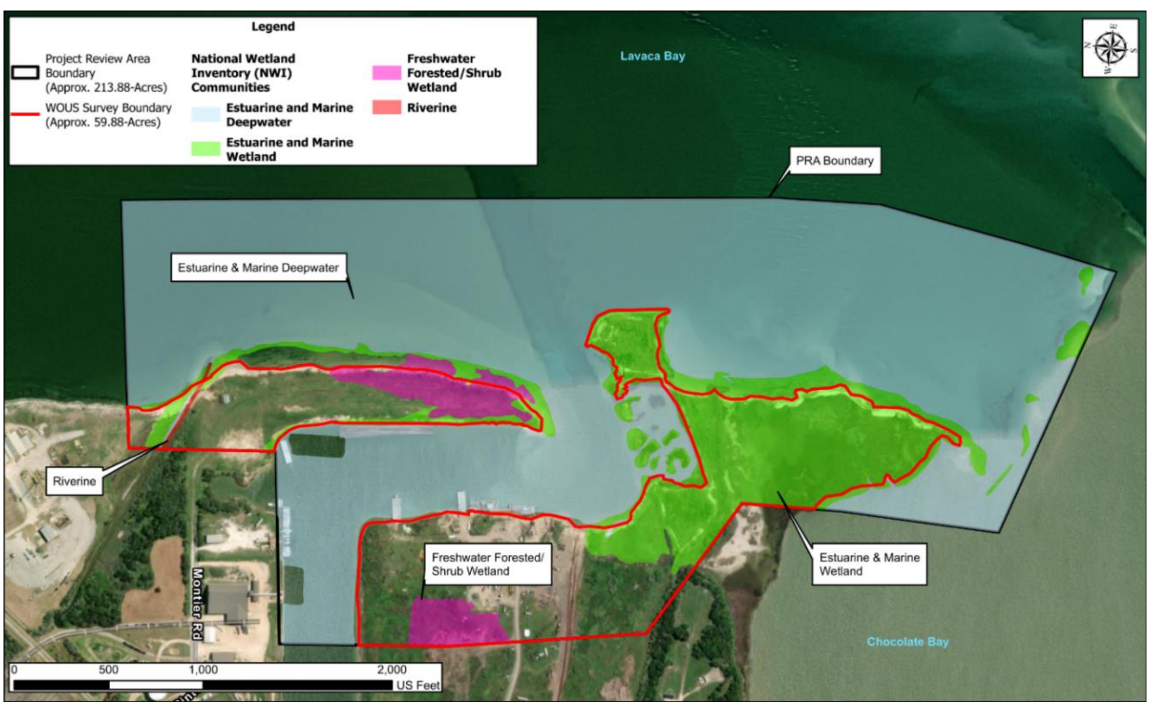

Figure 2.6: Habitat survey boundary and National Wetland Inventory (NWI) data.

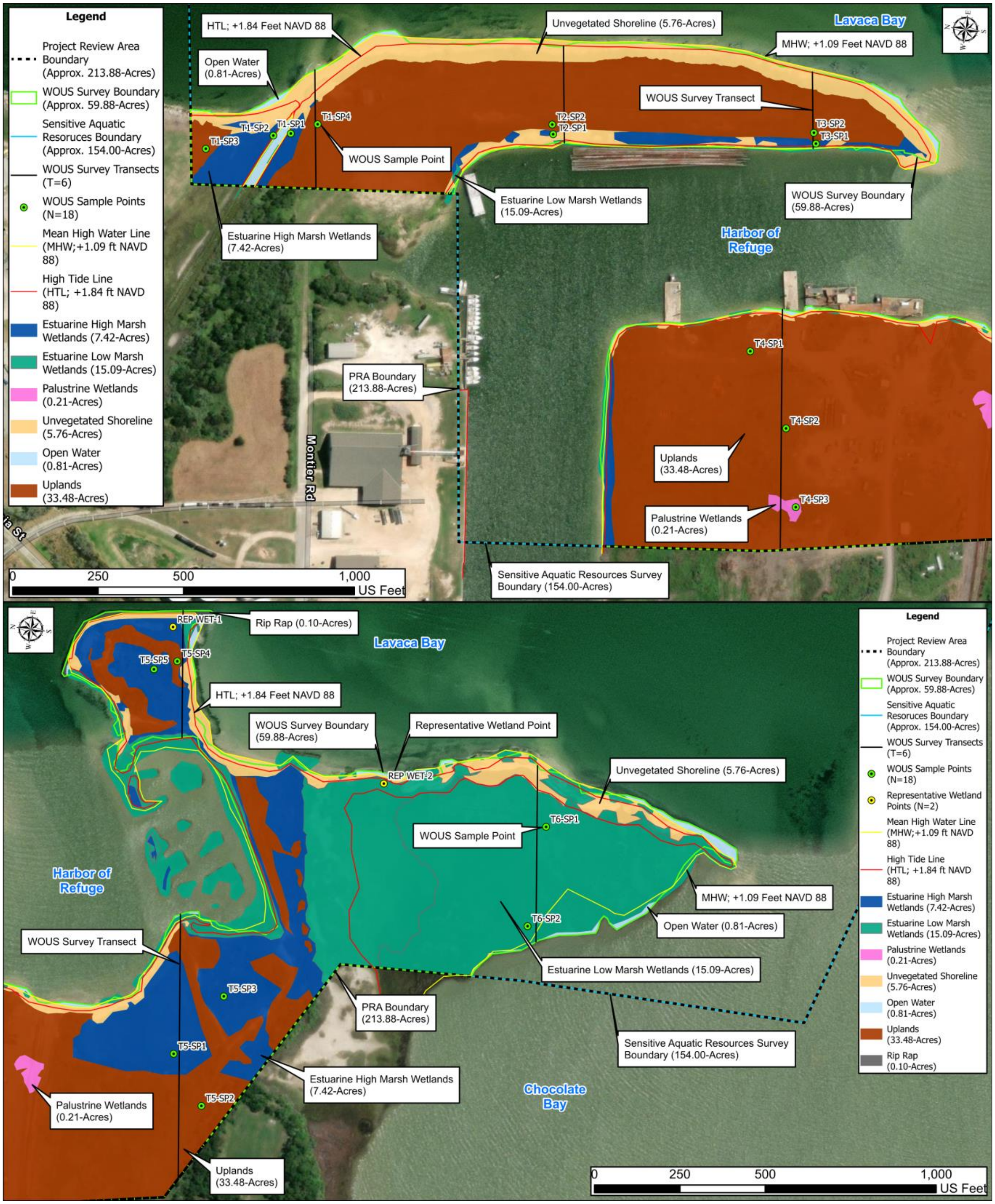

Figure 2.7: Overall habitat survey results.

Figure 2.8: Sensitive oyster bed habitat survey results.

2.5 Bathymetric Data

Bathymetric and topographic conditions were compiled using recent field data acquired by T. Baker Smith (TBS) during July 2022, as well as the Continuously Updated Digital Elevation Model (CuDEM), a product produced and regularly updated by NOAA. The 2024 TBS survey data includes bathymetric transects spaced at approximate 50 – 100 ft intervals along the project shoreline and is shown in Figure 2.9: T. Baker Smith survey data collected July 2022.

. The CuDEM has continuous coverage at a resolution of 3 m (~10 ft). The CuDEM bathymetry surface was used to extend coverage across Lavaca Bay and Matagorda Bay as shown in Figure 2.10. The CuDEM and newly collected survey data were merged to provide an accurate and continuous bathymetric surface for the project vicinity. The complete bathymetric surface was used in the numerical modeling of wind-waves.

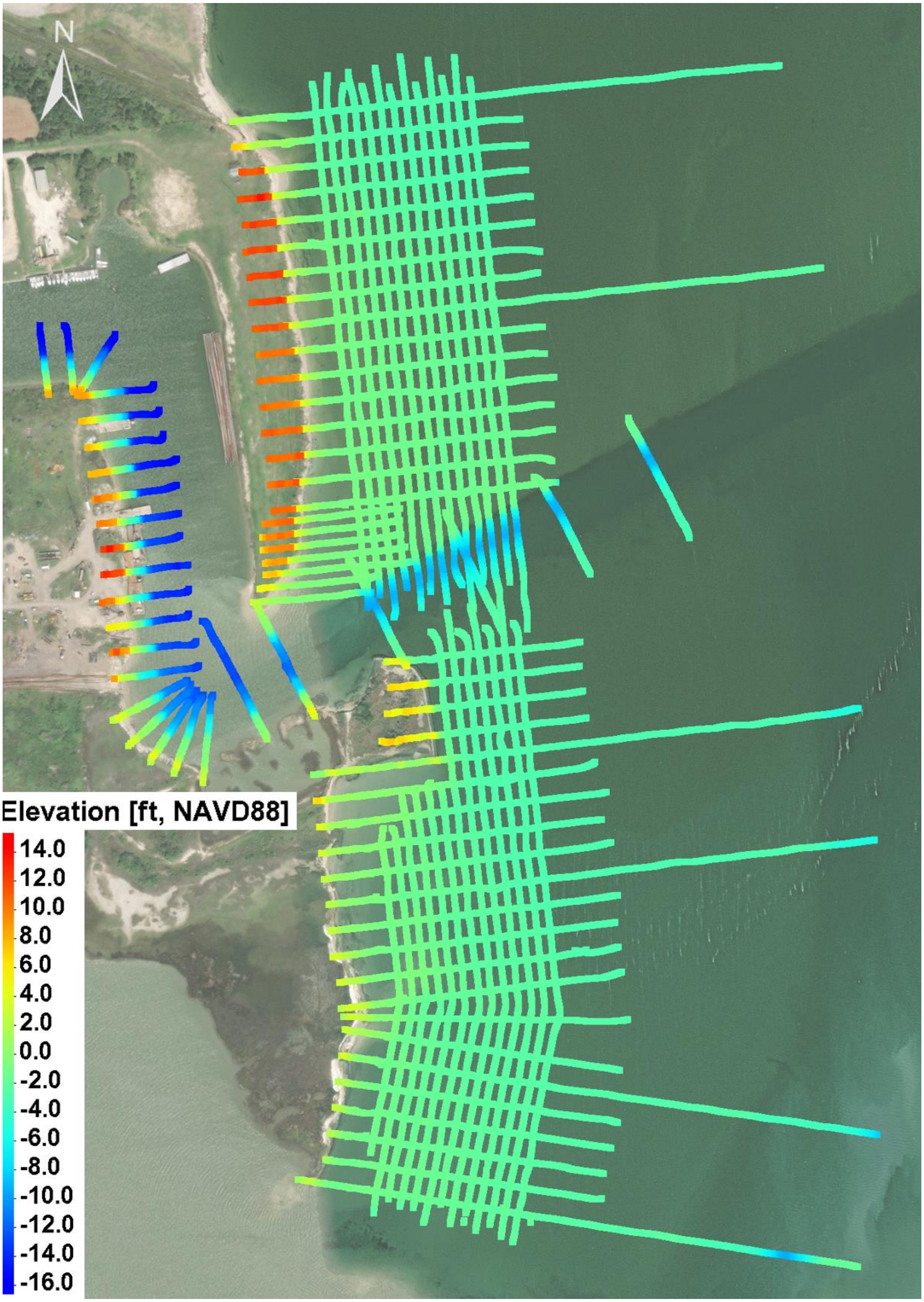

Figure 2.9: T. Baker Smith survey data collected July 2022.

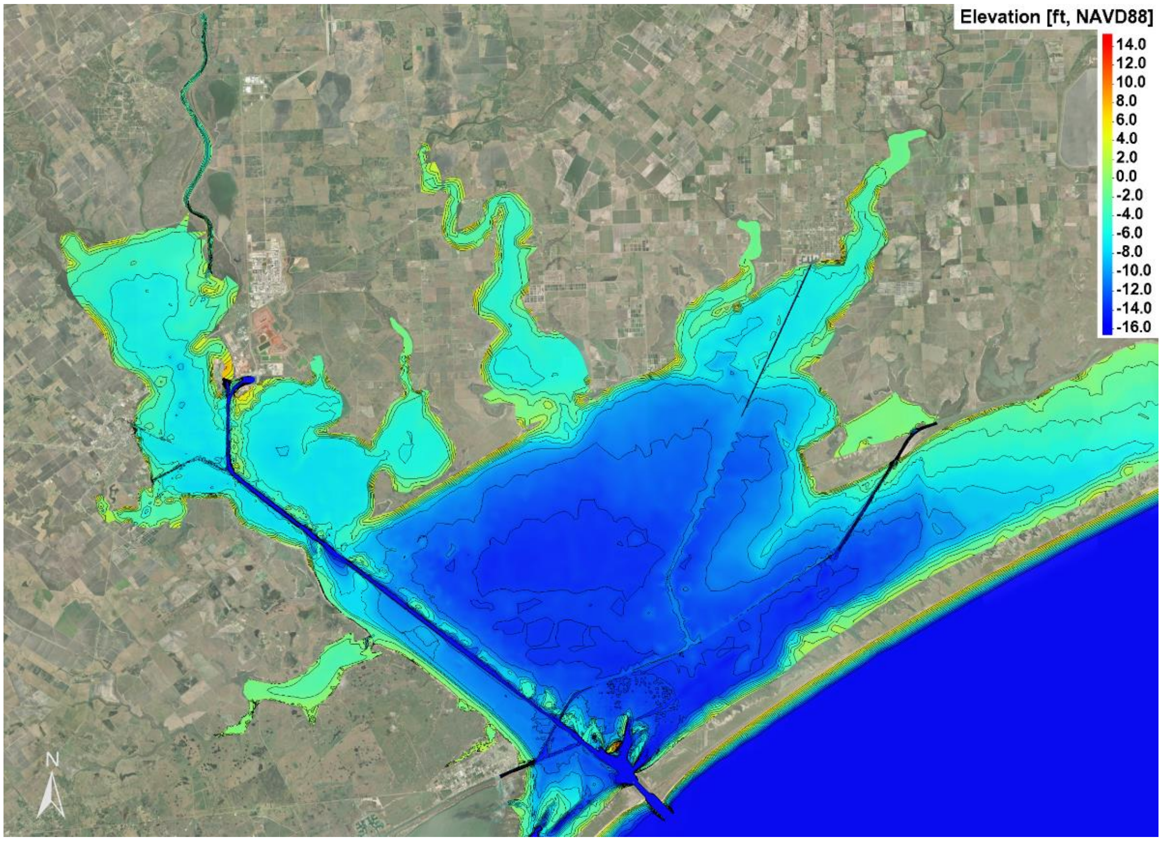

Figure 2.10: NOAA NCEI CuDEM bathymetry surface of Matagorda Bay.

3 Numerical Modeling

Numerical models were utilized to enhance the understanding of the coastal processes at the project site. Two nested SWAN models were constructed. The first, a lower resolution model, was designed to analyze the wave climate, calculate long-term alongshore sediment transport potential, and evaluate benefits of the alternatives to coastal ecosystems. The second, a high-resolution wave model, was set up to resolve the alternative coastal protection structures in detail and evaluate their performance in terms of wave attenuation. An XBeach model, which simulates the coastal morphodynamics of storm impacts, was created to analyze the performance of the alternatives in terms of storm-driven erosion protection. The overall goal of the numerical modeling was to evaluate the performance of the alternatives and establish a preferred alternative.

3.1 Wind-Wave Modeling

The primary consideration for structural design against coastal processes is the impact from tropical cyclones and the resulting wave attack. SWAN is a third-generation wave model utilized for obtaining realistic estimates of wave parameters in coastal areas due to wind-wave generation, propagation, and transformation. The model is based on the wave action balance equation with sources and sinks.

3.1.1 Long-term Wave Model Setup

A nesting approach was used to determine the wave climate of wind waves generated within Matagorda and Lavaca Bay and propagated to the project site. Waves were computed on a coarse grid ($dx = 800$ ft) for a larger region in Matagorda and Lavaca Bay (outer grid) and then computed on a finer grid ($dx = 164$ ft) for a smaller region comprising Chocolate Bay and parts of Lavaca Bay (inner grid) and then at an even finer resolution ($dx = 50$ ft) at the project site (inner grid L2) – see Figure 3.1 for the layout of the nested grid.

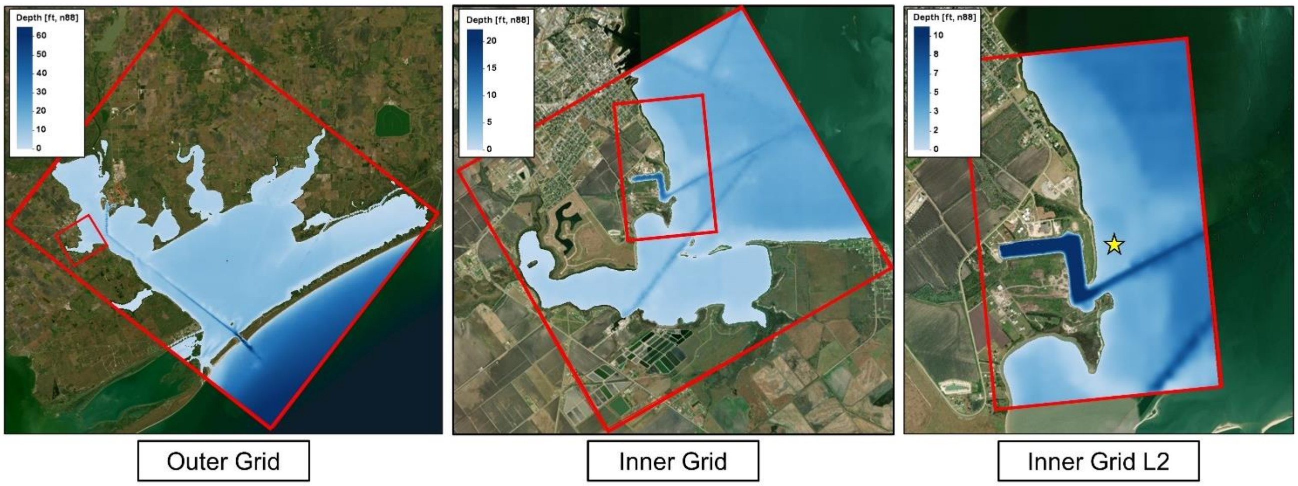

Figure 3.1: Long-term wave model grids. The red boxes indicate model domain boundaries. The yellow star indicates the output location for the wave climate analysis.

The long-term model was forced with hourly observed water levels and winds at the Port Lavaca NOAA gauge as shown in Figure 2.4. The SWAN model was run in non-stationary mode with a 1-hour timestep. The bathymetry was constructed by blending the TBS survey of the project site with NOAA's CuDEM project for the larger region.

3.1.2 Detailed Wave Model

To simulate detailed wave transformation processes at the project site, including the influence of the alternative coastal protection structures, a higher-resolution (compared to the long-term model setup in Section 3.1.1) wave model was created. SWAN was used for simulating the wave fields and a nested grid approach was again employed. Figure 3.2 shows the layout of the nested grids. The outer grid has a resolution of 330 ft, the inner grid has a resolution of 50 ft, and the level 2 inner grid has a resolution of 6 ft.

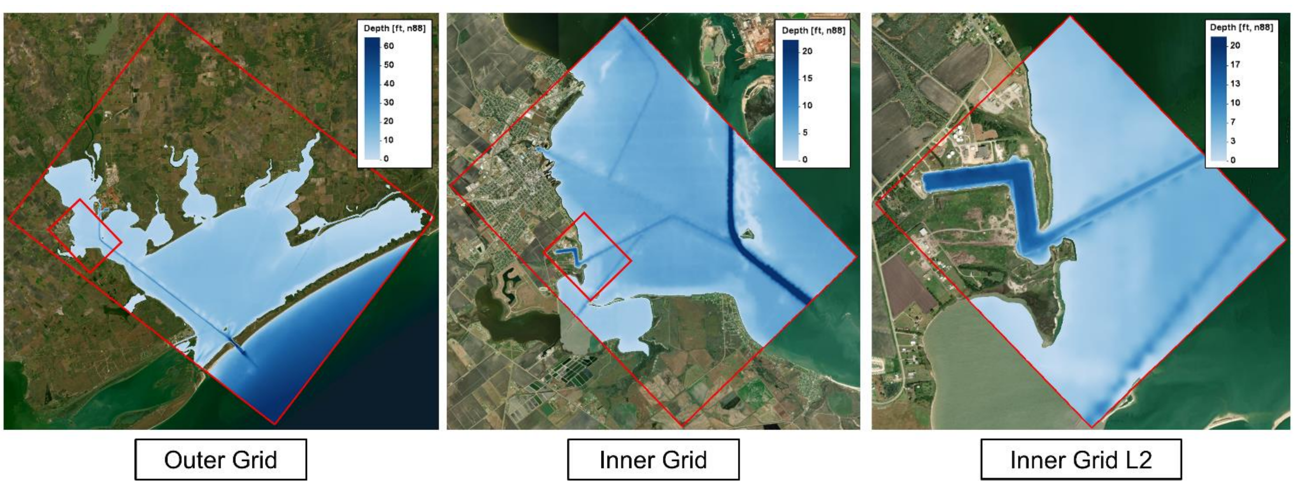

Figure 3.2: Detailed wave model grids. The red boxes indicate model domain boundaries.

3.1.3 Model Results

The wave modeling results were used in a number of applications. Besides the project site's wave climate (see Section 2.2.3) and extremal analysis (see Section 2.3.2), the detailed wave model was used to model alongshore transport potential (see Section 3.2), evaluate the performance of the alternatives in terms of wave energy attenuation (see Section 5.2), provide boundary conditions for the XBeach model (see Section 5.3), provide data for analyzing habitat suitability for marsh grass growth (see Section 5.4) and determine the stable stone size and cross-sectional geometry (see Section 4.1).

The layout of the alternatives and other relevant features of the wave modeling analysis are depicted in Figure 3.3. The colored arcs (labelled South, Inner, and North) indicate where wave heights are extracted when comparing the existing conditions to the wave conditions for each alternative. The dashed pink line is the one-dimensional XBeach model's domain where wave conditions are extracted at its seaward end and imposed as boundary conditions. Figure 3.3 should be used as a reference for interpreting the analysis of the wave modeling results.

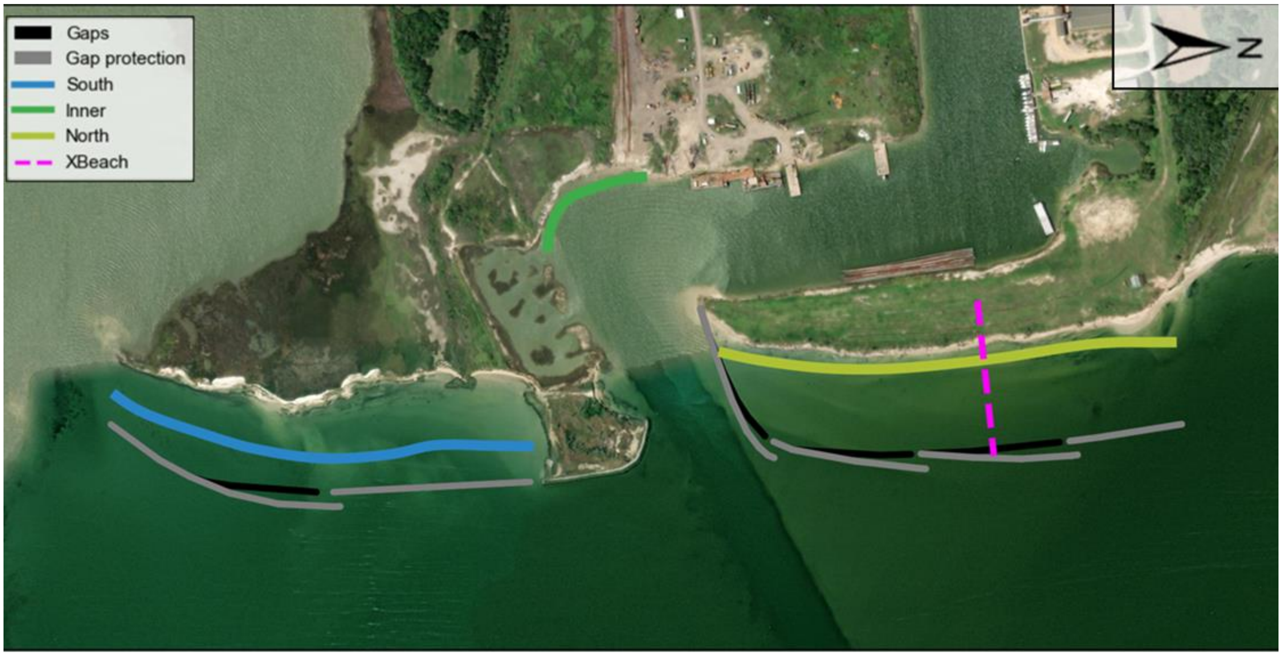

Figure 3.3: Detailed wave model features and alternative alignments. The black lines indicate the alignment for Alternatives 1, 3A, and 3B and the gray line represents Alternative 2 (see Section 5 for the alternative analysis.) The dashed pink line is the XBeach model domain (see Section 3.3 for the XBeach modeling). The remaining lines represent analysis arcs for the alternatives analysis.

3.2 Longshore Transport

The longshore sediment transport (LST) patterns at the project site were calculated using the CERC equation (USACE, 2002). Longshore sediment transport defines the rate at which sediment, i.e., sand and fines, is moved parallel to the shoreline. Changes in the rate along the shoreline lead to either shoreline erosion or accretion for increases and decreases in sediment transport, respectively. This is due to the conservation of sediment mass as either more or less sediment is transported out of a shoreline reach which is then either eroded from or deposited within the reach causing erosion or accretion, respectively.

Calculation of actual LST rates necessarily involve extensive field data collection and numerical model calibration, which is expensive and outside the scope of this project. However, calculating the potential for alongshore transport, which elides much of the complex sedimentological processes and is based instead on well understood hydrodynamic principles, is feasible and yields useful quantitative results. Therefore, in this analysis, alongshore sediment transport potential is estimated with the long-term wave model, and interpretation of the results provides a general picture of sediment flow along the project shoreline.

The CERC calculations depend on the wave height at breaking, local water depth, and breaking wave angle which were extracted along the project shoreline from the long-term wave model for the 8 years of the simulated wave hindcast. These values were used in the CERC calculations where northward transport was defined as positive by convention. The results of the calculations can be seen in Figure 3.4.

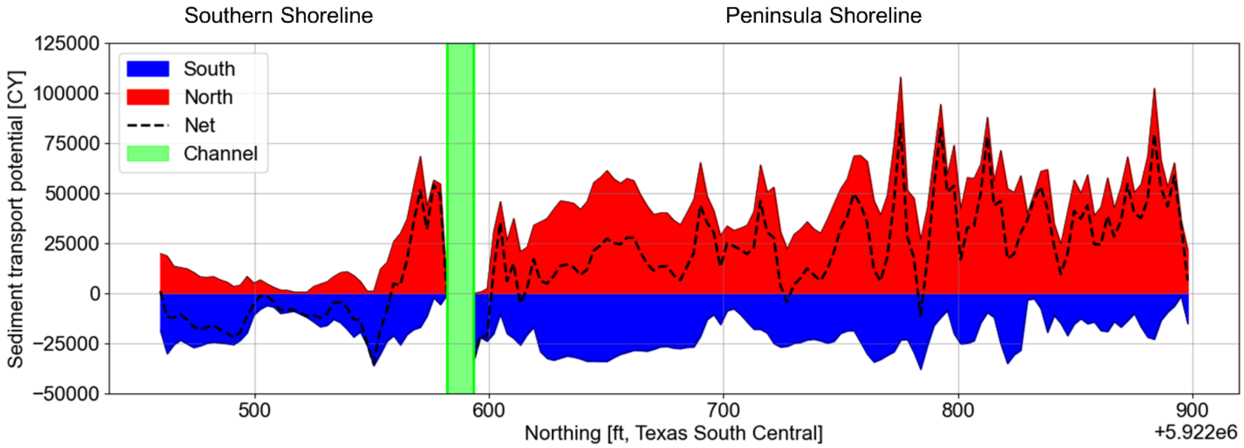

Figure 3.4: Integrated sediment transport potential from 2016 to 2024. The blue shaded area indicates transport to the south (left on the figure) and the red indicates northward transport (to the right). The black dashed line is the net transport potential. The channel shown is the one that enters the harbor (see Figure 4.1 for reach orientation at the project site).

The LST results show gross transport to both the north and south. The net transport is mostly to the north, suggesting that this is the direction of alongshore drift. However, north of the channel (indicated by the green band), the net transport is directed to the south. Additionally, the fact that there exists a degree of LST to the south, north of the channel, suggest that a jetty or other shore-normal structure is required to mitigate sedimentation in the channel.

3.3 XBeach Model

XBeach is a storm impact model which simulates wave- and current-driven sediment transport and the resulting morphological change. XBeach is used to simulate storm impacts in non-hydrostatic mode. Non-hydrostatic mode simulates individual waves, requiring very fine resolution in both space and time, which were approximately 9 inches and 0.003 seconds, respectively. A storm impact to a 1D cross-shore profile was simulated (see Figure 3.3 for the location) instead of simulating storm impacts to the entire project site as this would have been computationally unfeasible.

The cross-shore profile was selected along the northern reach (see Figure 3.3) where a steep bluff is critically eroding and vulnerable to storm impacts. The cross-section was interpolated from the bathymetric surface and can be seen in Figure 3.5. The offshore boundary is set to the location of the alternatives' alignment (see Section 4.2 and Figure 3.3). The nearshore profile is shallow with a mild slope and transitions into a shear bluff after approximately 50 ft of beach.

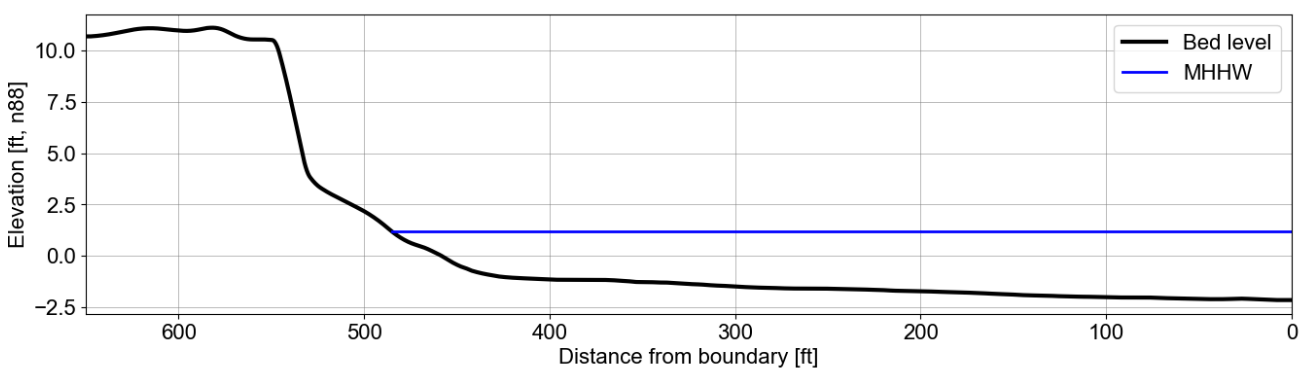

Figure 3.5: Bathymetric input for the 1D XBeach model. MHHW = mean high high-water.

Specification of the forcing conditions was synthesized from the extremal analysis (see Section 2.3) and an assumed hydrograph shape based on observed TC events in Lavaca Bay. The 25-year water level and significant wave height were used as the peak value of a Gaussian curve and the peak period was held constant at the 25-year extremal value. The storm duration was set to 24 hours and the shape of the Gaussian was determined by approximating observed TC hydrographs where the peak water level exceeded 3.28 ft NAVD88 at the Port Lavaca NOAA gauge (see Figure 3.6).

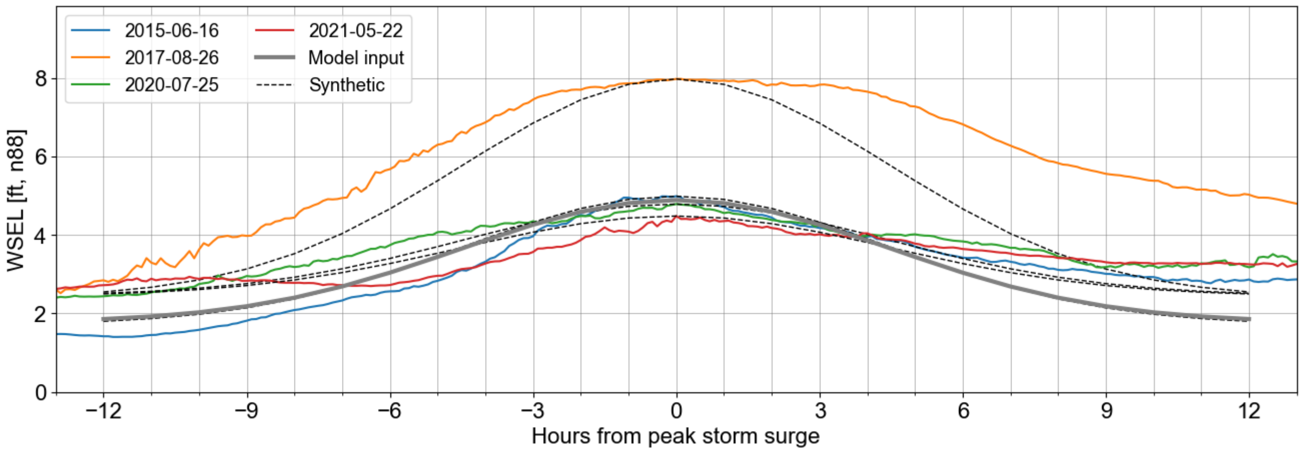

Figure 3.6: Model input, example synthetic, and observed storm hydrographs.

4 Alternatives Development

4.1 Common Design Elements

4.1.1 Rock Size

The stable rock size was selected to maintain stability of the structure during the 25-year design event and maximize constructability at the selected 3H:1V (seaward) and 1.5H:1V (landward) slopes. The stable rock size was selected in accordance with The Rock Manual (CIRIA, 2007) analytical stone sizing formulae. The stable median armor stone size was calculated to be 1.25 feet with a median weight of 350 lbs (calculated using the Van der Meer Shallow water stone sizing formula as presented in CIRIA, 2007). During final design, the stone size can be tuned to match the design requirements for stable stone size with commercially available stone gradations in the region.

4.1.2 Crest Elevation

The crest elevation of the structure controls the level of protection provided during daily conditions and storm events. Emergent breakwaters can provide low-energy environments in the lee of the structures, which protects landward facilities and shorelines from direct wave impact as well as encourages the survivability and future growth of seagrass and wetlands. Larger storm events typically have higher water levels that can submerge the breakwater and allow wave energy to impact areas landward of the breakwater. Therefore, structures with a higher crest elevation will help to reduce wave energy during both daily and storm conditions. However, the structure cost increases exponentially with increasing height. The crest elevation was designed to prioritize protection of the project area during daily conditions, while remaining structural stability during the design storm event.

The crest elevation for the breakwaters along the harbor peninsula and southern shoreline were evaluated at +5 feet NAVD88. Crest height may be evaluated further during preliminary design as it is a trade off between cost and performance. At this stage, we assumed a moderate crest elevation as a balance between the two. At a later stage of design, reducing the crest height may prove to be advantageous in a cost-benefit analysis, if the relevant reach of protected shoreline is assessed to be more resilient. For instance, the milder slope of the southern shoreline's beach and nearshore is typically understood to be less suspectable to scarping as the peninsula shoreline.

4.1.3 Slope

Side slopes for the rubble mound breakwater were selected to minimize the potential for scour, meet the necessary design life, and allow for a constructible rock size to be used. An appropriate structure slope was determined based on design wave heights and water levels at the project site. Generally, steeper slopes need larger rocks compared to shallower slopes to be stable for the same wave condition.

An appropriate slope of the breakwater face was selected to meet the 25-year design event and allow for a constructible rock size to be used in construction of the breakwater. The final seaward slope of the proposed structure is 3H:1V to keep the stone size in a reasonable range, and the landward slope is 1.5H:1V to reduce required volume of the structure.

4.2 Alternatives

All alternatives consist of approximately 300 linear feet of reef breakwater along the inner harbor shoreline, 1,300 linear feet of ACBM, and 1,900 linear feet of vegetated slope (Figure 4.1). Breakwater configurations along the harbor peninsula and southern shoreline are different between the alternatives.

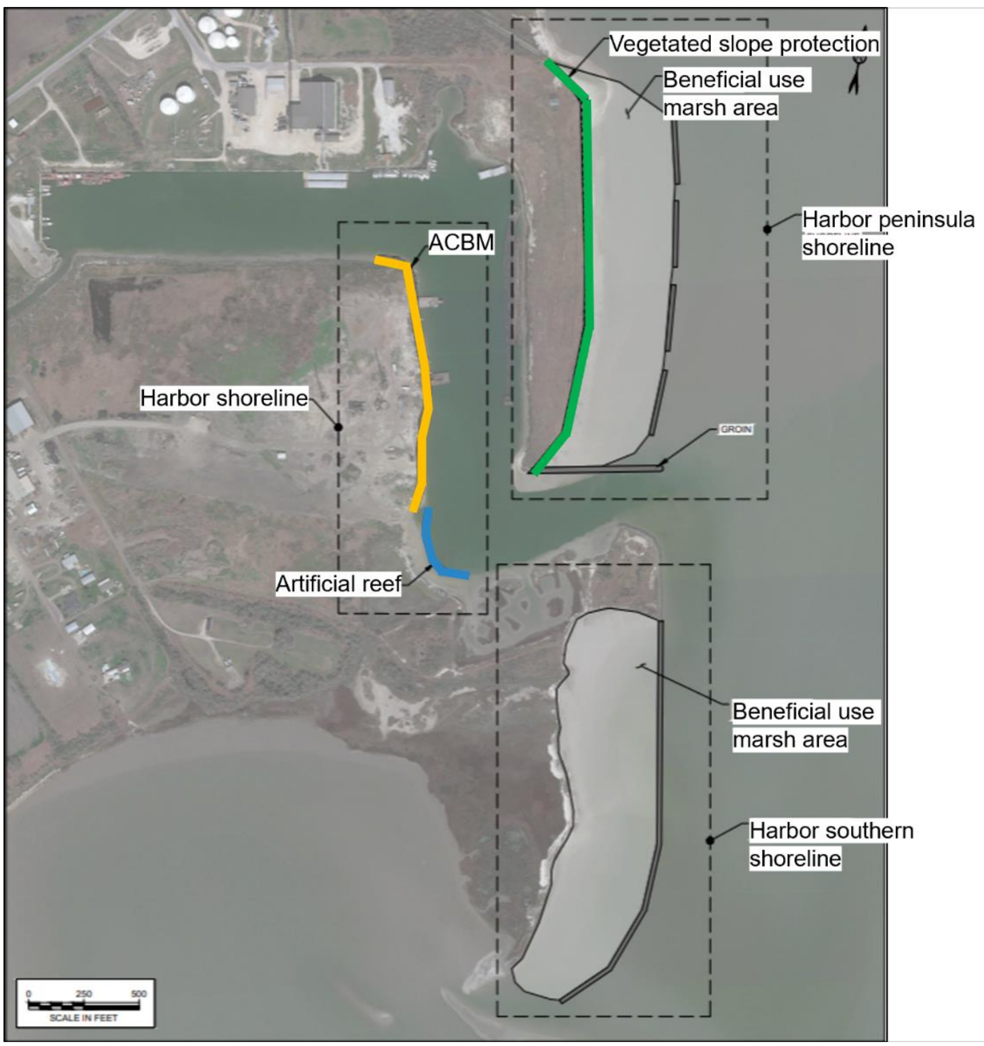

Figure 4.1: Concept design developed at project conception. Inner harbor shoreline reef breakwater (blue), ACBM (orange), and vegetated slope (green) for all alternatives.

4.2.1 Alternative 1

Alternative 1 breakwater configuration (Figure 4.2) along the harbor peninsula and southern shoreline consists of 6 segmented rubble mound breakwaters for a total length of approximately 3,645 linear feet.

Figure 4.2: Alternative 1 breakwater configuration. Rubble mound breakwater is shown as the magenta line.

4.2.2 Alternative 2

Alternative 2 breakwater configuration (Figure 4.3) along the harbor peninsula and southern shoreline consists of 6 shingled segmented rubble mound breakwaters for a total length of approximately 3,977 linear feet. The shingled configuration serves as gap protection and can be refined in 30% design as needed. The shingle orientation is to protect against the largest waves which come from the SE direction.

Figure 4.3: Alternative 2 breakwater configuration. Rubble mound breakwater is shown as the magenta line.

4.2.3 Alternative 3

4.2.3.1 Concept A

Alternative 3A breakwater configuration (Figure 4.4) consists of 4 segmented rubble mound breakwaters in the north (approximately 2,045 linear feet) and 2 segmented artificial reef unit breakwaters in the south (approximately 1,600 linear feet). The artificial reef is assuming 3 rows of 6.5’ tall Wave Attenuation Devices (WAD) for construction.

Figure 4.4: Alternative 3A breakwater configuration. Rubble mound breakwater is shown as the magenta line and artificial reef unit breakwater is shown as the blue line.

4.2.3.2 Concept B

Alternative 3B breakwater configuration (Figure 4.5) consists of 4 segmented rubble mound breakwaters in the north, 1 rubble mound breakwater in the south, and 1 reef unit breakwater in the south. The rubble mound breakwater length is approximately 2,795 linear feet and the artificial reef unit breakwater length is approximately 850 linear feet.

Figure 4.5: Alternative 3B breakwater configuration. Rubble mound breakwater is shown as the magenta line and artificial reef unit breakwater is shown as the blue line.

5 Alternatives Analysis

5.1 Performance Criteria

The project should meet a set of performance criteria developed during conceptual design. Based on internal discussions of the project goals and objectives, the project performance criteria were determined to be:

1. Attenuate waves for the design storm event.
2. Protect again storm-driven erosion.
3. Transmitted wave height is within tolerable limits for marsh vegetation colonization.

5.2 Wave Attenuation

The wave modeling approach developed in Section 3.1 was applied to each of the alternatives to quantify their wave energy attenuation performance during extreme events. Waves were simulated for each set of return period conditions (water level and wind speed) from four wind directions and the maximum wave parameters along each alternative's alignment (see Section 4.2) was extracted. These wave conditions, along with design parameters for the alternatives, were used to estimate the wave attenuation afforded by the alternative coastal protection structure per return period condition.

The alternative coastal protection structures were then built into the SWAN model as obstructions with prescribed wave attenuation coefficients (k_t). Wave attenuation is the ratio of the transmitted wave height to the incident wave height, $k_t = H_t/H_i$, where H_t is the transmitted wave height and H_i is the incident wave height. The method of Buccino and Calabrese (2007) was used to calculate k_t for the rubble mound structures and Mott MacDonald (2019) was used to calculate k_t for the artificial reef units. With the alternatives in place, the return period conditions were again simulated at the four wind directions.

Figure 2.1 shows Alternative 1 during 1-year conditions form the east-southeast (105°). The lack of gap protection is obvious in the wave field leeward of the breakwater, where significant wave energy is allowed to propagate towards the shoreline. Figure 5.2 shows Alternative 2 with the same forcing conditions. In contrast to Alternative 1, the gap protection in Alternative 2 is effective in attenuating waves from this direction. This can clearly be seen in the difference plot in Figure 5.2.

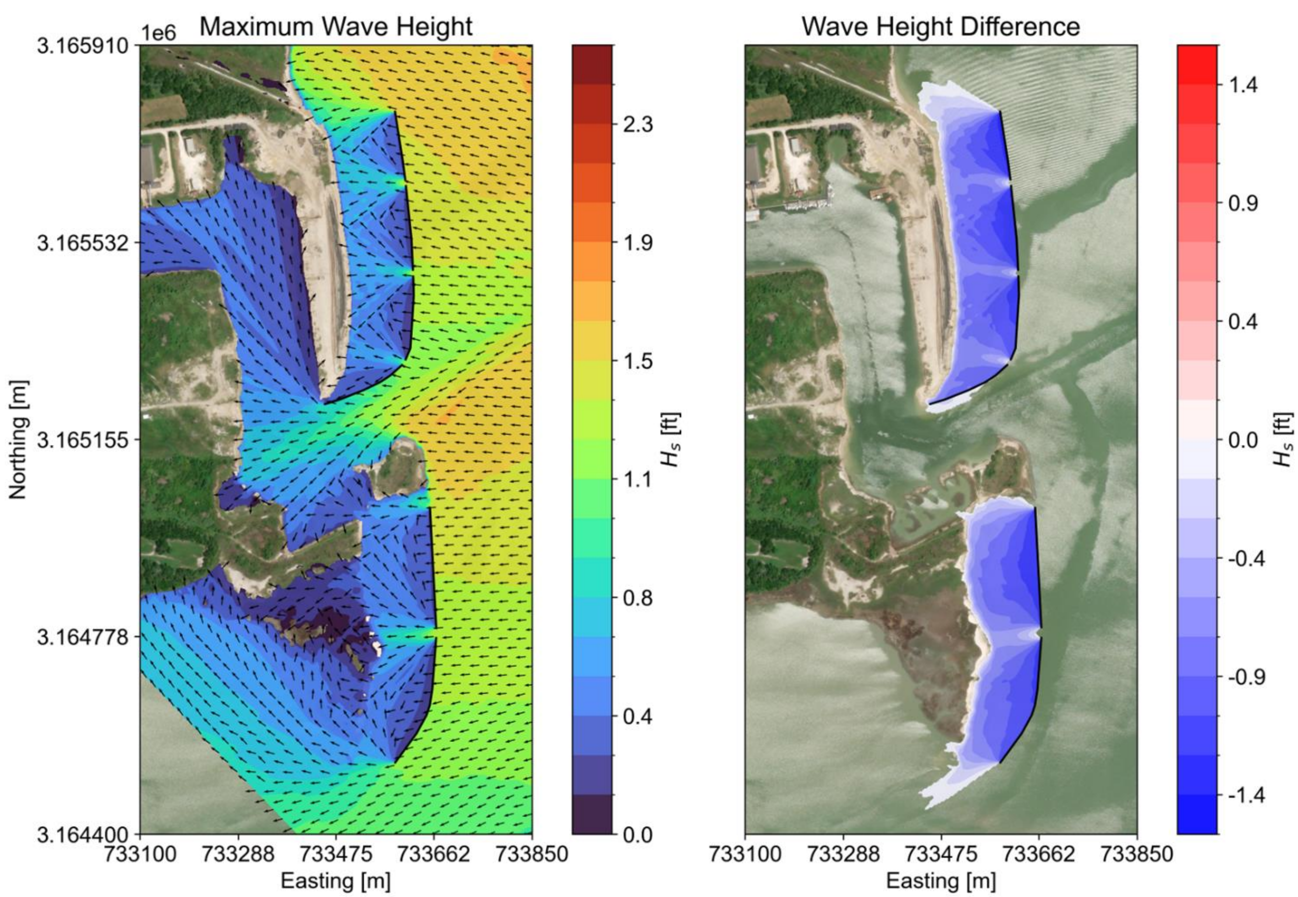

Figure 5.1: Simulated wave height and difference with existing conditions for Alternative 1 with the 1-year return period conditions from the east-southeast (105°).

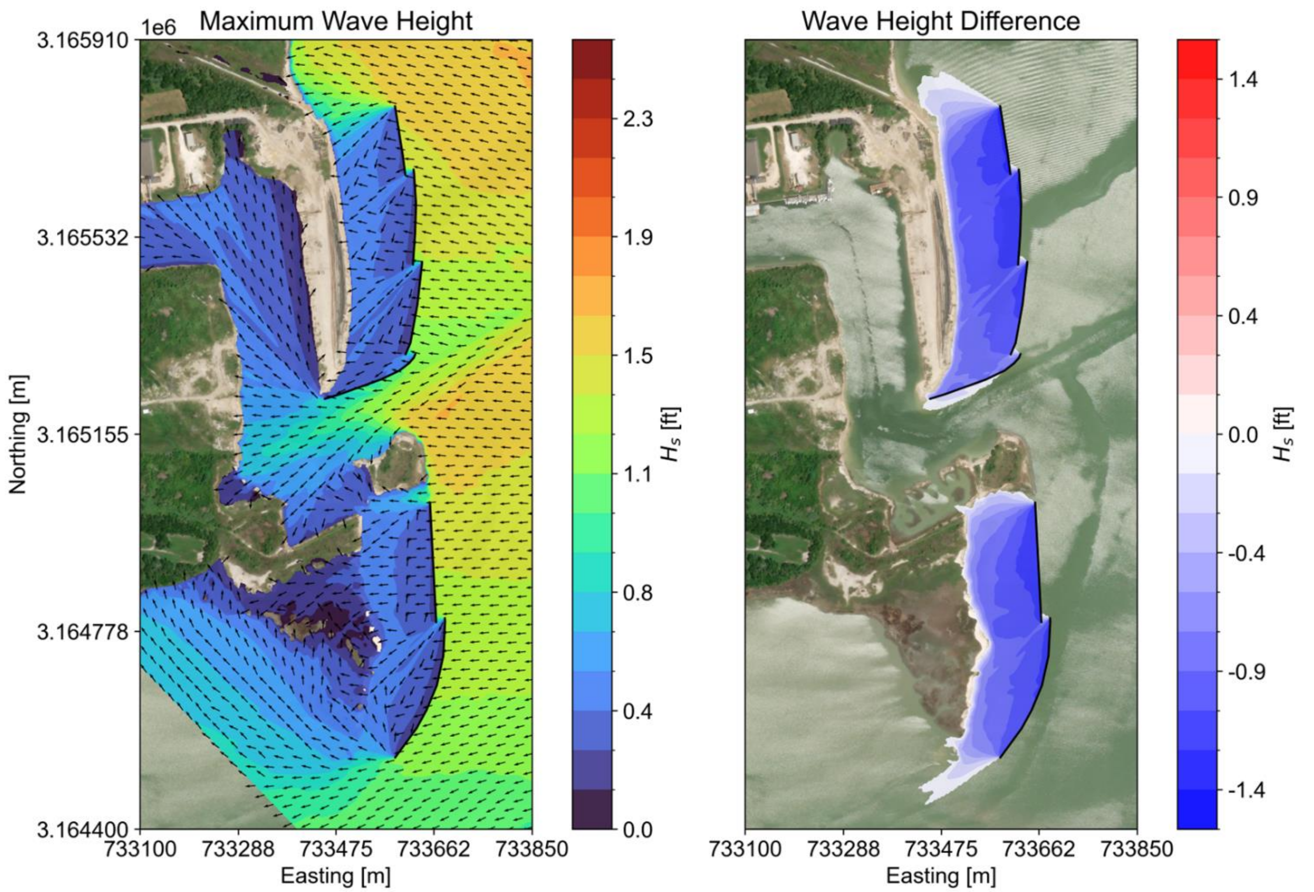

Figure 5.2: Simulated wave height and difference with existing conditions for Alternative 2 with the 1-year return period conditions from the east-southeast (105°).

Figure 5.3 shows Alternative 2 with 1-year forcing conditions out of the north-northeast (10°). Waves from this direction can freely propagate behind the structure. The gap protection requires adjustment in order to address this wave condition. Further refinement, which is designed to shelter the project shoreline from northern and northeastern waves, is planned in the next phase of design (see Section 5.6 for recommended refinements). For completeness, the maximum transmitted wave height and difference from existing conditions for all the simulation results for the all the return period conditions can be found in Appendix A.1.

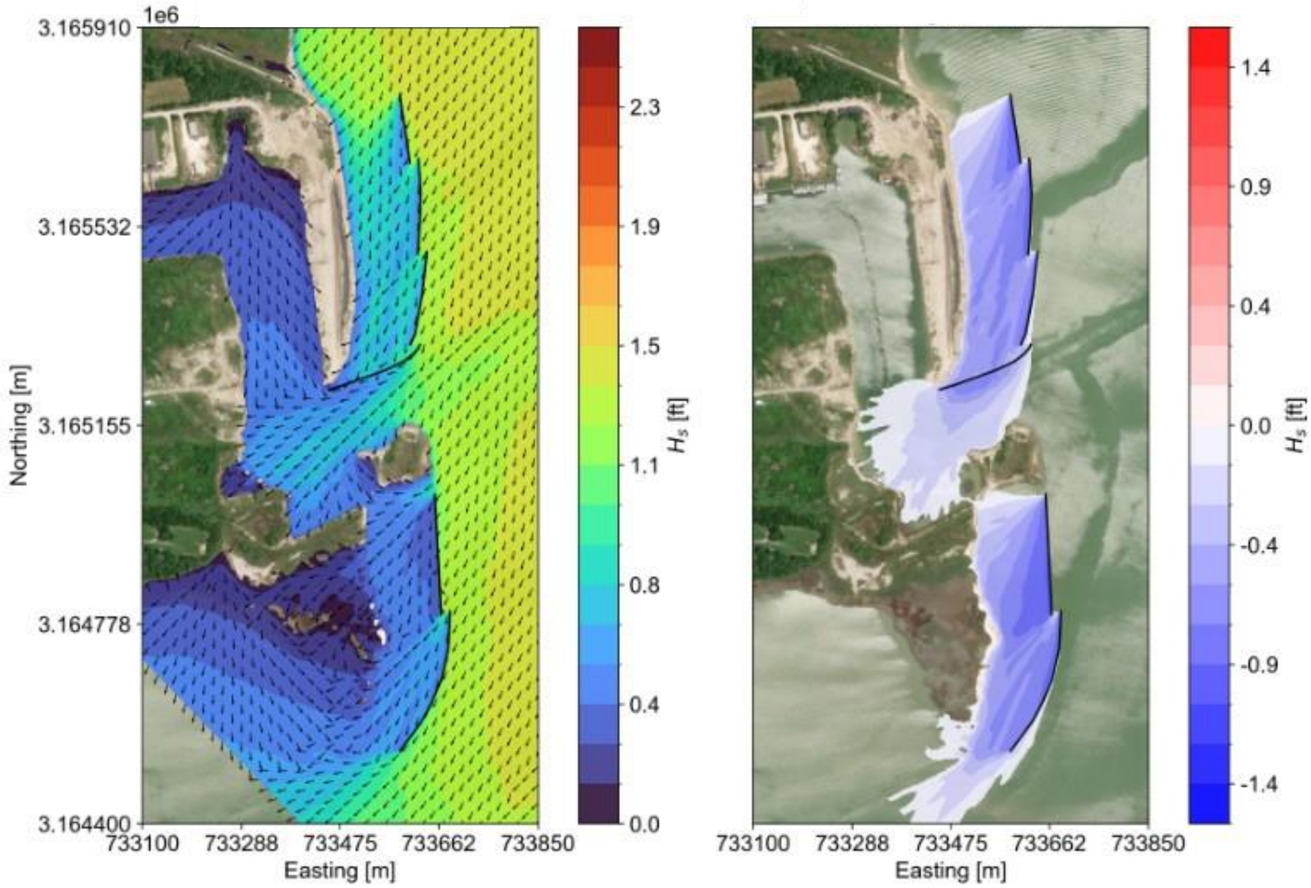

Figure 5.3: Simulated wave height and difference with existing conditions for Alternative 2 with the 1-year return period conditions from the north-northeast (10°).

There is significant spatial variability along the project shoreline in terms of wave height for each alternative. In order to visualize this variability, wave height was extracted along three reaches (see Figure 3.3 for the reach extents). The range of wave heights due to differences in wave direction and alternative alignment layout are plotted as a band of possible wave heights. See Figure 5.4 through Figure 5.7 for the alternatives' wave attenuation performance for the 1- (annual cold front), 5- (large cold front), 25- and 100-year return period conditions, respectively. Table 5.1 shows the wave height reduction for each alternative under the 25-year design case (results in this table are associated with Figure 5.6, remaining tabulated results can be found in Table A.1).

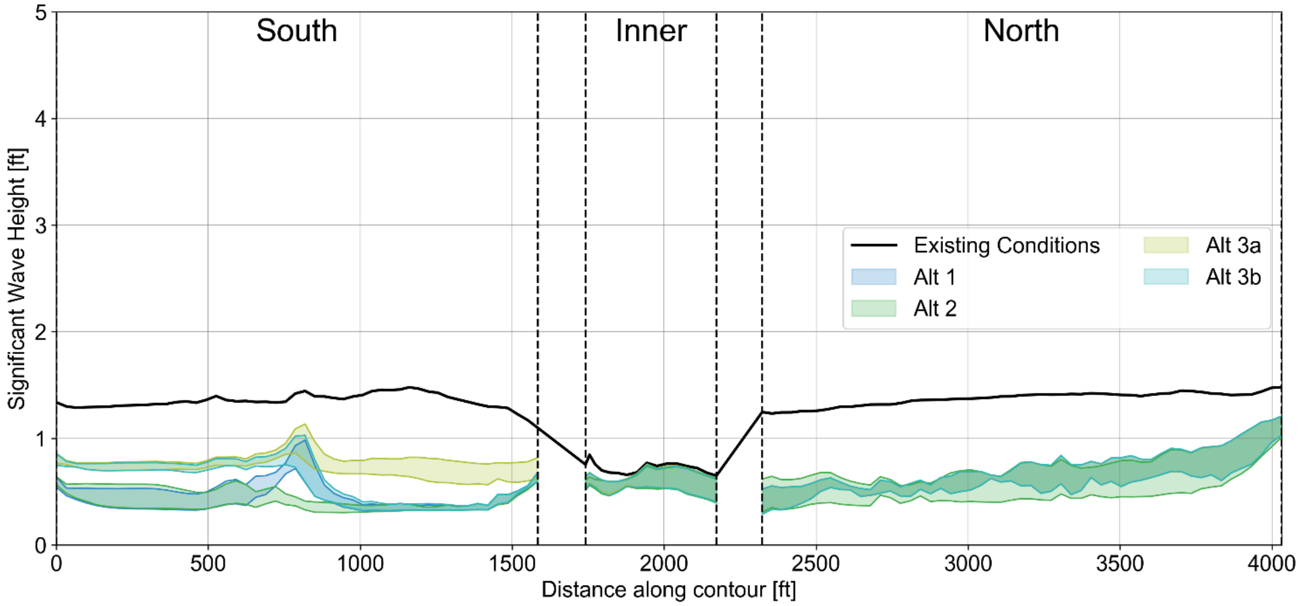

Figure 5.4: Wave height distribution along shoreline reaches for 1-year return period conditions. The maximum wave height for existing conditions is shown as a black line.

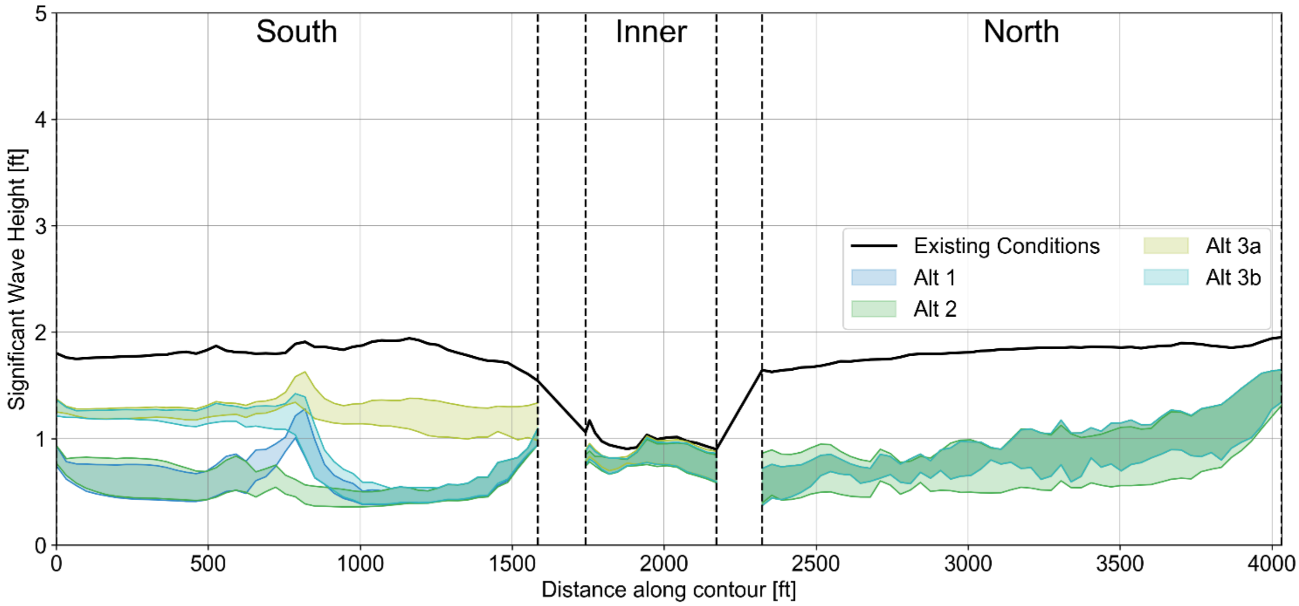

Figure 5.5: Wave height distribution along shoreline reaches for 5-year return period conditions. The maximum wave height for existing conditions is shown as a black line.

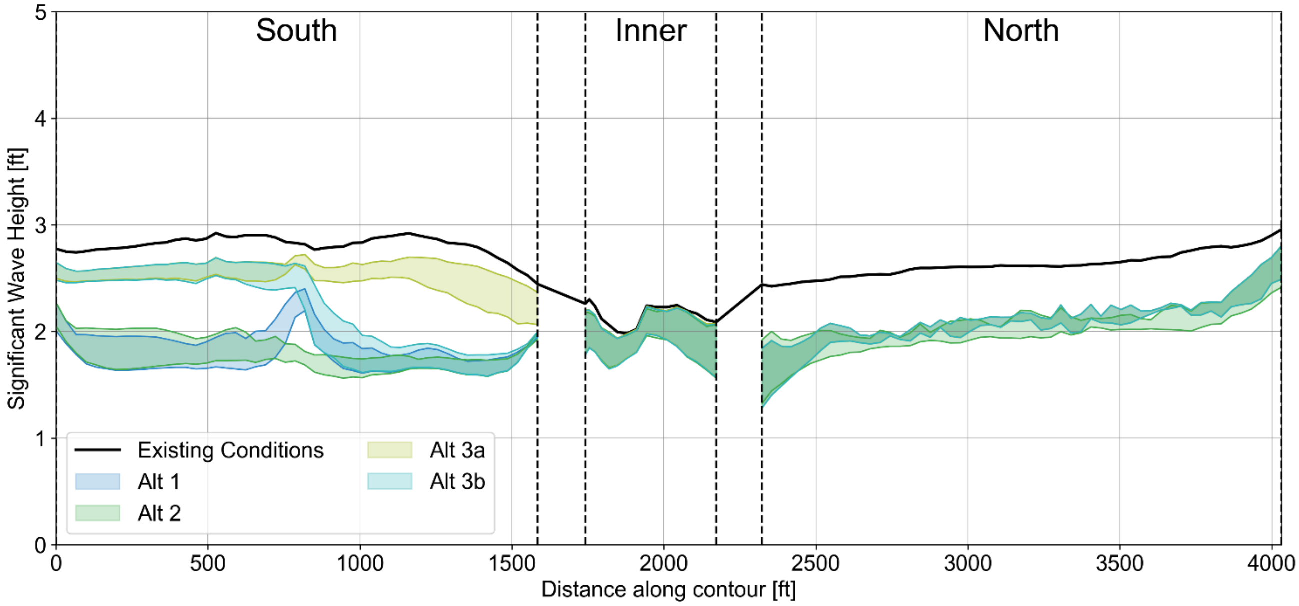

Figure 5.6: Wave height distribution along shoreline reaches for 25-year return period conditions. The maximum wave height for existing conditions is shown as a black line.

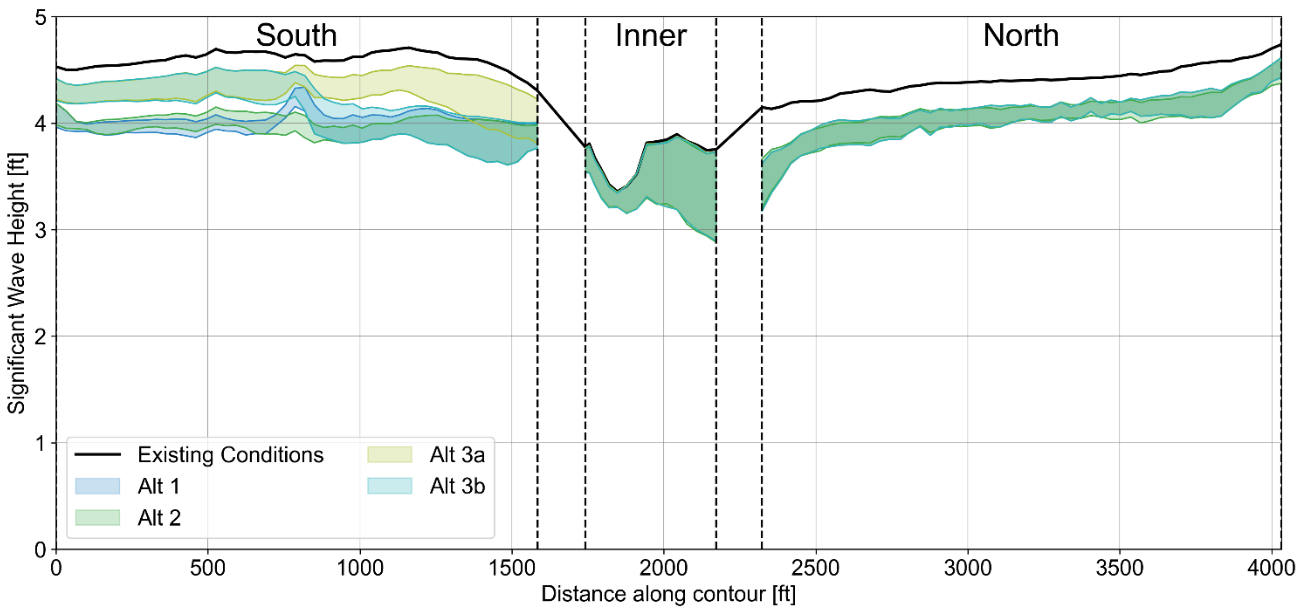

Figure 5.7: Wave height distribution along shoreline reaches for 100-year return period conditions. The maximum wave height for existing conditions is shown as a black line.

Table 5.1: Alternative wave height reduction from Existing Conditions for 25-year design case.

Alternative	Reach	Absolute Hs diff [ft]	% Difference
Alternative 1	South	-0.88	-31.1
Alternative 2	South	-0.92	-32.8
Alternative 3A	South	-0.19	-6.7
Alternative 3B	South	-0.52	-18.4
Alternative 1	Inner	-0.05	-2.1
Alternative 2	Inner	-0.05	-2.4
Alternative 3A	Inner	-0.04	-1.8
Alternative 3B	Inner	-0.05	-2.1
Alternative 1	North	-0.88	-18.1
Alternative 2	North	-0.92	-18.7
Alternative 3A	North	-0.19	-18.1
Alternative 3B	North	-0.52	-18.1

There is a reduction in wave height across all the return period conditions for all the alternatives, indicating that each alternative would offer some level of protection. In general, the protection afforded by the alternatives decreases with increasing return period as increased water levels relative to the structure's crest elevation reduce wave attenuation.

Alternative 1 and Alternative 2 show similar performance, but Alternative 2 has a lower lower-bound within the wave height band due the gap protection. However, the upper-bound between the two alternatives is similar. This difference in protection appears marginal for the extremal return period conditions but may result in larger net benefits to shoreline protection through time.

The major difference in wave attenuation performance can be seen on the southern reach with Alternative 3A and 3B. Alternative 3A uses artificial reef units (ARU) for the entire southern reach and Alternative 3B uses them for the southern segment (see Section 4.2.3). Artificial reef units which offer increased habitat and aesthetics at the cost of reduced wave reduction performance for most cases. For the lower return periods ($Tr = 1\ to\ 5$ years), the ARU performs only slightly worse than the rubble mounds, but the difference in performance decreases for larger return periods (Table A.1).

5.3 Erosion Protection

The XBeach modeling approach developed in Section 3.3 was applied to Alternative 1 and 2 to quantify their performance in terms of storm-driven erosion control. Note that this analysis does not consider the effects of a vegetated shoreline or a marsh creation cell. The maximum and average horizontal, linear bluff retreat within the simulations was calculated above the +4 ft NAVD88 contour. The results are presented in Table 5.2. In terms of bluff retreat, the alternatives both offer a good level of protection, reducing the average retreat by 50% and maximum by 33%. Both Alternative 1 and 2 exhibit the same performance, according to these metrics, within the uncertainty of the model.

Table 5.2: XBeach linear bluff retreat during a 25-year event.

Case	Average linear retreat [ft]	Maximum linear retreat [ft]
Existing Conditions	3.0	4.9
Alternative 1	1.5 (50%)	3.3 (33%)
Alternative 2	1.5 (50%)	3.3 (33%)

The volume of material above MHHW after the simulation was calculated for existing conditions and Alternatives 1 and 2 to determine erosion (ft^3/ft) for a 25-year event (Table 5.3). Figure 5.8 plots the cross-shore profile for the initial bed level (black dashed line) prior to the 25-year event and the bed level after the 25-year event for the alternative (blue line) and existing conditions (green line).

Table 5.3: XBeach erosion rates for 25-year event.

Case	Erosion [ft^3/ft]
Existing Conditions	14.5
Alternative 1	1.0 (-93%)
Alternative 2	1.4 (-91%)

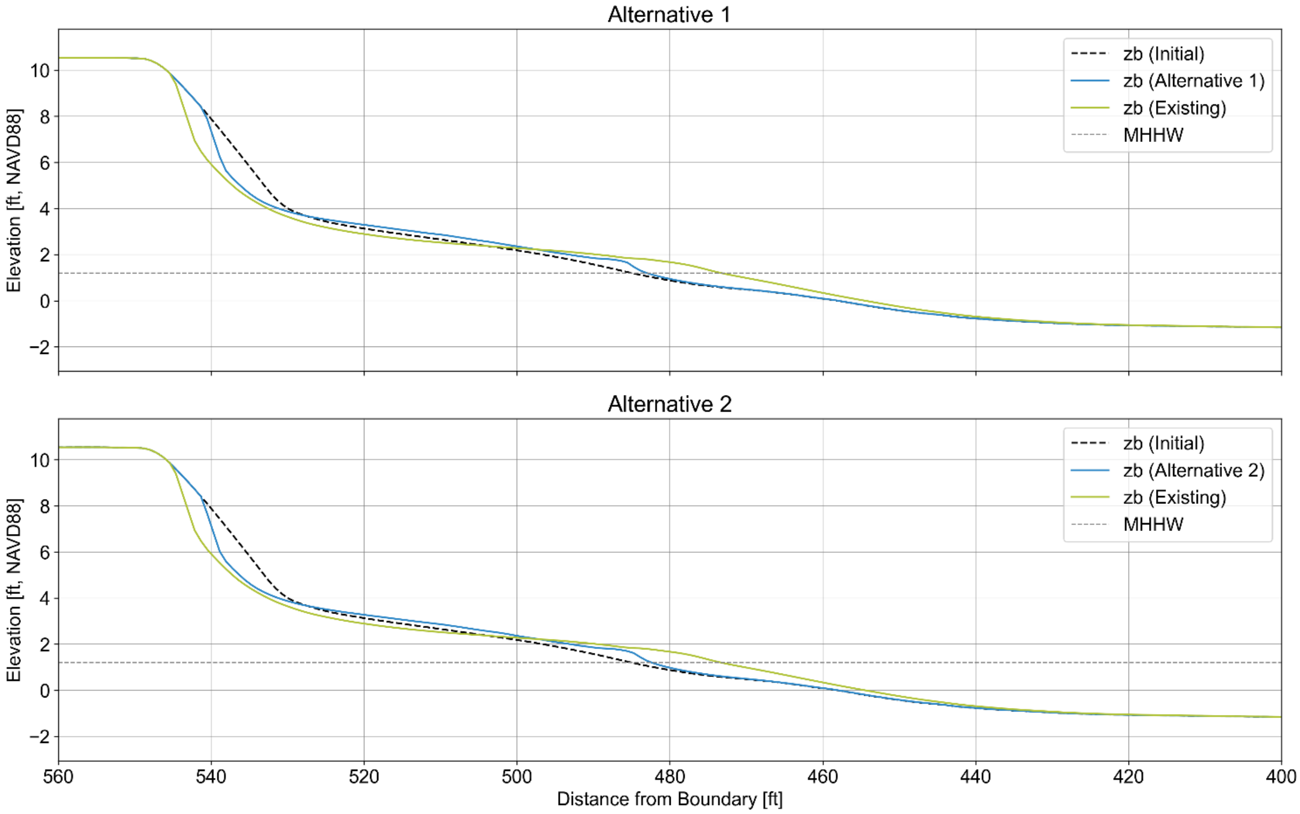

Figure 5.8: XBeach profile model results for Alternatives 1 and 2.

From Table 5.3 and Figure 5.8, both Alternative 1 and 2 perform well in retaining sediment above MHHW during a 25-year event. Both alternatives are able to reduce linear erosion by 50% compared to existing conditions.

5.4 Marsh Creation

The alternative's performance in terms of marsh creation primarily depends on the coastal protection's capacity for reducing incident wave heights to be within the threshold for typical inter-tidal vegetation colonization.

Figure 5.9 plots the transmitted wave height for existing conditions (incident waves), rubble mound breakwater, and oyster reef breakwater. The gray polygon represents the Roland and Douglass (2005) tolerance for marsh vegetation. The existing condition is near the upper limits of the tolerance, which indicates the existing marsh is in an intermediate zone, tending towards erosion. Introducing a rubble mound or oyster reef breakwater would result in conditions well below the tolerance limits, which suggests new marsh would be stable under typical wave conditions. Note at the upper end of the exceedance curve (Figure 5.9, 99-100%) are associated with infrequent extremal events. For these events, the marsh is typically resilient since it is likely submerged and not exposed to direct wave attack.

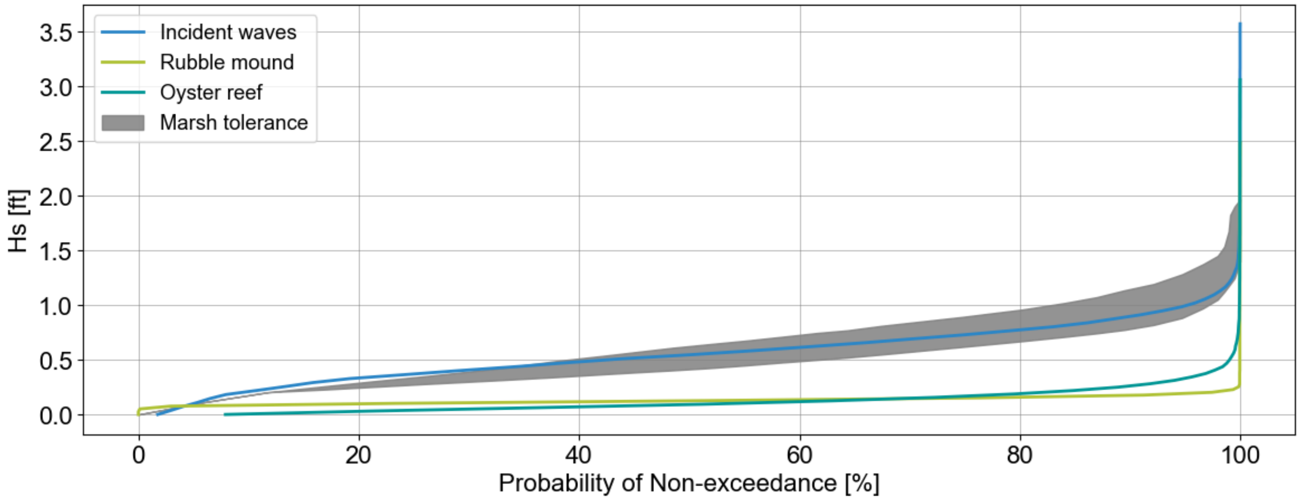

Figure 5.9: Marsh tolerance of wave height with transmitted wave heights from a rubble mound structure and an artificial oyster reef.

5.5 Discussion & Performance Assessment

5.5.1 Cost

Costs for the rock and reef breakwaters, ACBM, and vegetated slope were established through contractor estimates and contractor bids from similar recent projects from 2020 onward. These costs were adjusted for inflation using the U.S. Bureau of Labor Statistics inflation calculator. All costs are in 2024 dollars. Assuming a 2026 construction timeline, the costs can be adjusted for the future, assuming 3% annual inflation, by multiplying the total cost by 1.061. Costs do not include final engineering, bidding phase support, construction oversight, or construction administration. Actual quantities at time of construction may vary due to change in site conditions. Unit costs fluctuate and may be different at the time of construction.

Alternative 3A has the highest cost at approximately $11.6 million while Alternative 1 – 5ft Crest Elevation Rock Breakwater has the lowest cost at approximately $8.8 million. The cost estimate breakdown for each alternative is shown in Table 5.4 through Table 5.7.

Table 5.4: Alternative 1 – 5ft Crest Elevation Rock Breakwater

Item Description	Quantity	Unit	Unit Cost	Total
Mobilization and Demobilization	1	LS	$662,830.00	$662,830.00
Construction Surveying	5	EA	$20,000.00	$100,000.00
Environmental Protection	1	LS	$46,080.00	$46,080.00
Rock Breakwaters - Stone	28,336	TON	$176.00	$4,987,136.00
Rock Breakwaters - Geotextile	14,378	SY	$14.00	$201,292.00
Inner Harbor Reef Breakwater - Units	300	LF	$850.00	$255,000.00
Inner Harbor Reef Breakwater - Bedding Stone	223	TON	$99.00	$22,077.00
Inner Harbor Reef Breakwater - Geotextile	289	SY	$14.00	$4,046.00
ACBM	13,942	SF	$23.00	$320,666.00
ACBM - Geotextile	1,550	SY	$14.00	$21,700.00
ACBM – Bedding Stone	403	TON	$99.00	$39,897.00
Vegetated Slope – Soil Bags	19,539	SF	$32.00	$625,248.00
Vegetated Slope – Planting	0.5	AC	$10,209.00	$5,104.50
			Contingency (20%)	$1,458,215.30
			Total	$8,749,291.80

Table 5.5: Alternative 2 – 5ft Crest Elevation Rock Breakwater with gap protection

Item Description	Quantity	Unit	Unit Cost	Total
Mobilization and Demobilization	1	LS	$710,440.00	$710,440.00
Construction Surveying	5	EA	$20,000.00	$100,000.00
Environmental Protection	1	LS	$49,390.00	$49,390.00
Rock Breakwaters - Stone	30,918	TON	$176.00	$5,441,568.00
Rock Breakwaters - Geotextile	15,688	SY	$14.00	$219,632.00
Inner Harbor Reef Breakwater - Units	300	LF	$850.00	$255,000.00
Inner Harbor Reef Breakwater - Bedding Stone	223	TON	$99.00	$22,077.00
Inner Harbor Reef Breakwater - Geotextile	289	SY	$14.00	$4,046.00
ACBM	13,942	SF	$23.00	$320,666.00
ACBM - Geotextile	1,550	SY	$14.00	$21,700.00
ACBM – Bedding Stone	403	TON	$99.00	$39,897.00
Vegetated Slope – Soil Bags	19,539	SF	$32.00	$625,248.00
Vegetated Slope – Planting	0.5	AC	$10,209.00	$5,104.50
			Contingency (20%)	$1,562,953.70
			Total	$9,377,722.20

Table 5.6: Alternative 3A – 5ft Crest Elevation Northern Rock Breakwater, Southern Reef Breakwater

Item Description	Quantity	Unit	Unit Cost	Total
Mobilization and Demobilization	1	LS	$878,570.00	$878,570.00
Construction Surveying	5	EA	$20,000.00	$100,000.00
Environmental Protection	1	LS	$61,080.00	$61,080.00
Rock Breakwaters - Stone	15,898	TON	$176.00	$2,798,048.00
Rock Breakwaters - Geotextile	8,067	SY	$14.00	$112,938.00
Southern Reef Breakwater - Units	4,800	LF	$850.00	$4,080,000.00
Southern Reef Breakwater - Bedding Stone	2,779	TON	$99.00	$275,121.00
Southern Reef Breakwater - Geotextile	4,623	SY	$14.00	$64,722.00
Inner Harbor Reef Breakwater - Units	300	LF	$850.00	$255,000.00
Inner Harbor Reef Breakwater - Bedding Stone	223	TON	$99.00	$22,077.00
Inner Harbor Reef Breakwater - Geotextile	289	SY	$14.00	$4,046.00
ACBM	13,942	SF	$23.00	$320,666.00
ACBM - Geotextile	1,550	SY	$14.00	$21,700.00
ACBM – Bedding Stone	403	TON	$99.00	$39,897.00
Vegetated Slope – Soil Bags	19,539	SF	$32.00	$625,248.00
Vegetated Slope – Planting	0.5	AC	$10,209.00	$5,104.50
			Contingency (20%)	$1,932,843.50
			Total	$11,597,061.00

Table 5.7: Alternative 3B – 5ft Crest Elevation Northern Rock Breakwater, Partial Southern Reef Breakwater

Item Description	Quantity	Unit	Unit Cost	Total
Mobilization and Demobilization	1	LS	$777,490.00	$777,490.00
Construction Surveying	5	EA	$20,000.00	$100,000.00
Environmental Protection	1	LS	$54,050.00	$54,050.00
Rock Breakwaters - Stone	21,729	TON	$176.00	$3,824,304.00
Rock Breakwaters - Geotextile	11,025	SY	$14.00	$154,350.00
Southern Reef Breakwater - Units	2,550	LF	$850.00	$2,167,500.00
Southern Reef Breakwater - Bedding Stone	1,480	TON	$99.00	$146,520.00
Southern Reef Breakwater - Geotextile	2,456	SY	$14.00	$34,384.00
Inner Harbor Reef Breakwater - Units	300	LF	$850.00	$255,000.00
Inner Harbor Reef Breakwater - Bedding Stone	223	TON	$99.00	$22,077.00
Inner Harbor Reef Breakwater - Geotextile	289	SY	$14.00	$4,046.00
ACBM	13,942	SF	$23.00	$320,666.00
ACBM - Geotextile	1,550	SY	$14.00	$21,700.00
ACBM – Bedding Stone	403	TON	$99.00	$39,897.00
Vegetated Slope – Soil Bags	19,539	SF	$32.00	$625,248.00
Vegetated Slope – Planting	0.5	AC	$10,209.00	$5,104.50
			Contingency (20%)	$1,710,467.30
			Total	$10,262,803.80

5.5.2 Alternatives Evaluation Matrix

Alternatives analysis is performed with the goal of objectively choosing a preferred alternative. The criteria and weights used in the evaluation matrix are shown in Table 5.8. These weights are based on our estimation of the relative importance, but the weights should be adjusted by project stakeholders to match the importance to the stakeholders. The preferred alternative was selected based on these metrics.

Table 5.8: Evaluation matrix criteria.

Criteria	Weight
Performance – Wave Reduction	40%
Cost	30%
Habitat Benefits	25%
Aesthetics	5%

Performance – Wave Reduction includes the alternative's ability to attenuate waves (Section 5.2) and provide erosion protection (Section 5.3). Cost reflects the alternative's estimated cost for construction (Section 5.5.1). Habitat Benefits were evaluated by the alternative's potential to create suitable conditions for marsh vegetation (Section 5.4) and provide oyster habitat through the use of ARUs. Aesthetics were rated from the perspective that the use of ARUs is more aesthetically pleasing and considered more natural than rubble mound breakwater. See scoring for each alternative in Figure 5.10.

Criteria	Performance - Wave Reduction		Cost		Habitat Benefits		Aesthetics		
Weighted Factor:	40%		30%		25%		5%		100%
Alternatives	**Score**	**Weighted Score**	**Score**	**Weighted Score**	**Score**	**Weighted Score**	**Score**	**Weighted Score**	**TOTAL SCORE**
Alternative 1 - 5ft Crest Elevation Rock Breakwater	4.0	1.6	5.0	1.5	4.0	1.0	3.0	0.2	4.25
Alternative 2 - 5ft Crest Elevation Rock Breakwater	5.0	2.0	4.0	1.2	4.0	1.0	3.0	0.2	4.35
Alt 3A - 5ft Crest Elevation Northern Rock Breakwater, Southern Reef Breakwater	3.0	1.2	2.0	0.6	5.0	1.3	5.0	0.3	3.30
Alt 3B - 5ft Crest Elevation Northern Rock Breakwater, Partial Southern Reef Breakwater	3.0	1.2	3.0	0.9	5.0	1.3	4.0	0.2	3.55

Figure 5.10: Evaluation matrix.

5.5.3 Performance Assessment

As discussed in Section 5.2, along the southern project shoreline, the gaps in Alternatives 1, 3A, and 3B allow more wave energy to propagate towards the shoreline when compared to Alternative 2, which implements shingled gap protection. Furthermore, the artificial reef units (ARUs) used in Alternatives 3A and 3B reduced performance compared to rubble mound breakwater options in Alternatives 1 and 2. Along the northern project shoreline, Alternative 2 performs slightly better than Alternative 1 due to the shingled gap protection. The shingled gap protection can be refined in 30% design to optimize wave attenuation.

In terms of storm-driven erosion control, Alternative 1 and 2 performed well in retaining sediment and were able to reduce erosion by over 90% compared to existing conditions (Section 5.3).

When evaluating the wave tolerance for marsh creation (Section 5.4), both rubble mound and ARU breakwater alternatives would result in conditions well below the marsh tolerance limits, which suggests the marsh would be stable.

5.6 Alternative Selection

The preferred alternative is Alternative 2 with a modification and enhancement of the protection. Alternative 2 performs sufficiently well to attenuate waves, provide erosion control, and create conditions for marsh vegetation while maintaining a moderate estimated construction cost. However, it was identified that significant wave energy could penetrate to the northern project shoreline from the north.

Alternative 4 modifies Alternative 2 to protect against north and northeast wave conditions by extending the gap protection overlap and bending the northernmost breakwater toward the shoreline to protect against waves from the north-northeast direction. Additionally, the effect of increasing the crest elevation was also evaluated by lifting the northern rock breakwater to a +6 ft NAVD88 crest elevation. If additional budget is available, adding artificial reef units in the southernmost reach near the breach location may be worth considering. For clarification, Alternative 4A and 4B refer to the +5 ft and + 6 ft crest elevation along the northern harbor peninsula reach, respectively.

5.6.1 Alternative 4A (enhanced gap protection)

Alternative 4A breakwater configuration (Figure 5.11) along the harbor peninsula and southern shoreline consists of 6 segmented rubble mound breakwaters for a total length of approximately 4,493 linear feet. The shingled configuration from Alternative 2 was modified in Alternative 4A by extending the gap protection overlap.

Figure 5.11: Alternative 4A breakwater configuration. Rubble mound breakwater is shown as the magenta line.

The enhanced gap protection of Alternative 4A effectively shelters the entire project shoreline. This can be seen in Figure 5.12 and Figure 5.13 which shows the 1-year wave conditions for the North-northeast (10°) and East-southeast (105°) directions, respectively. The wave fields behind the protection are significantly more attenuated than Alternative 1 or 2 (see Section 5.2). The wave modeling results for the remaining wave conditions are located in Appendix A.1.

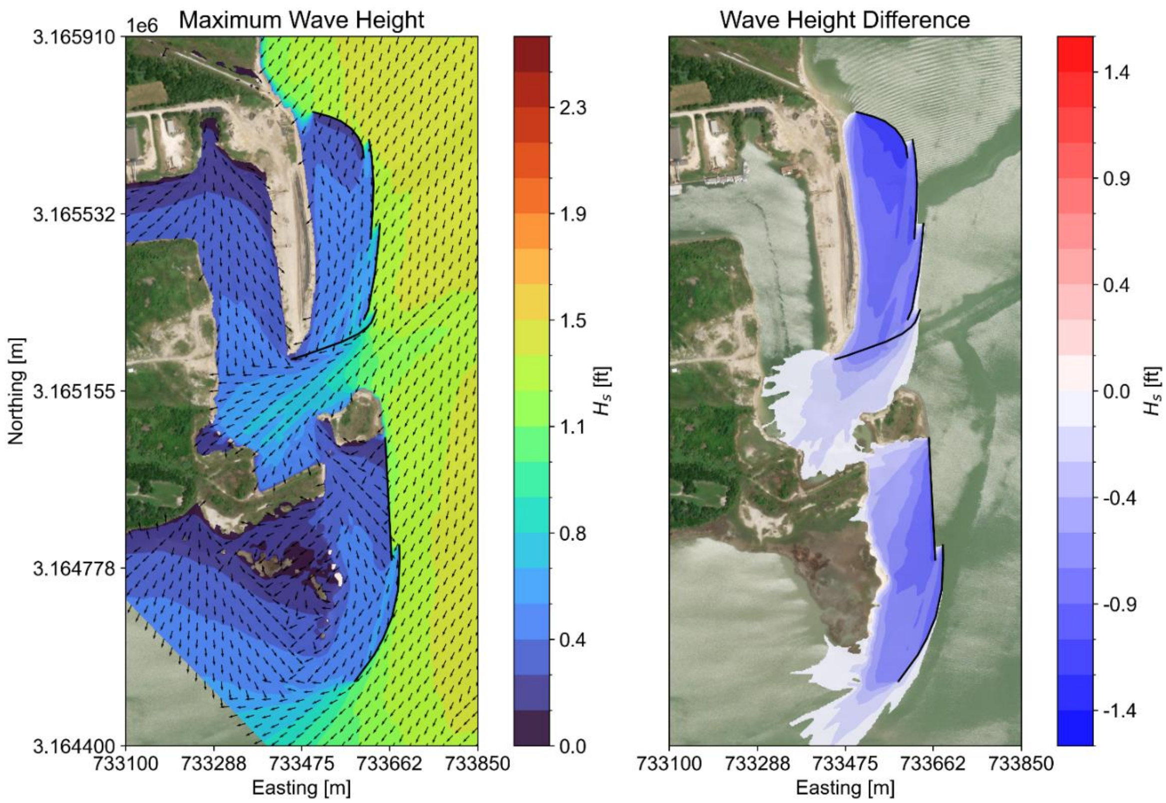

Figure 5.12: Simulated wave height and difference with existing conditions for Alternative 4A with the 1-year return period conditions from the north-northeast (10°).

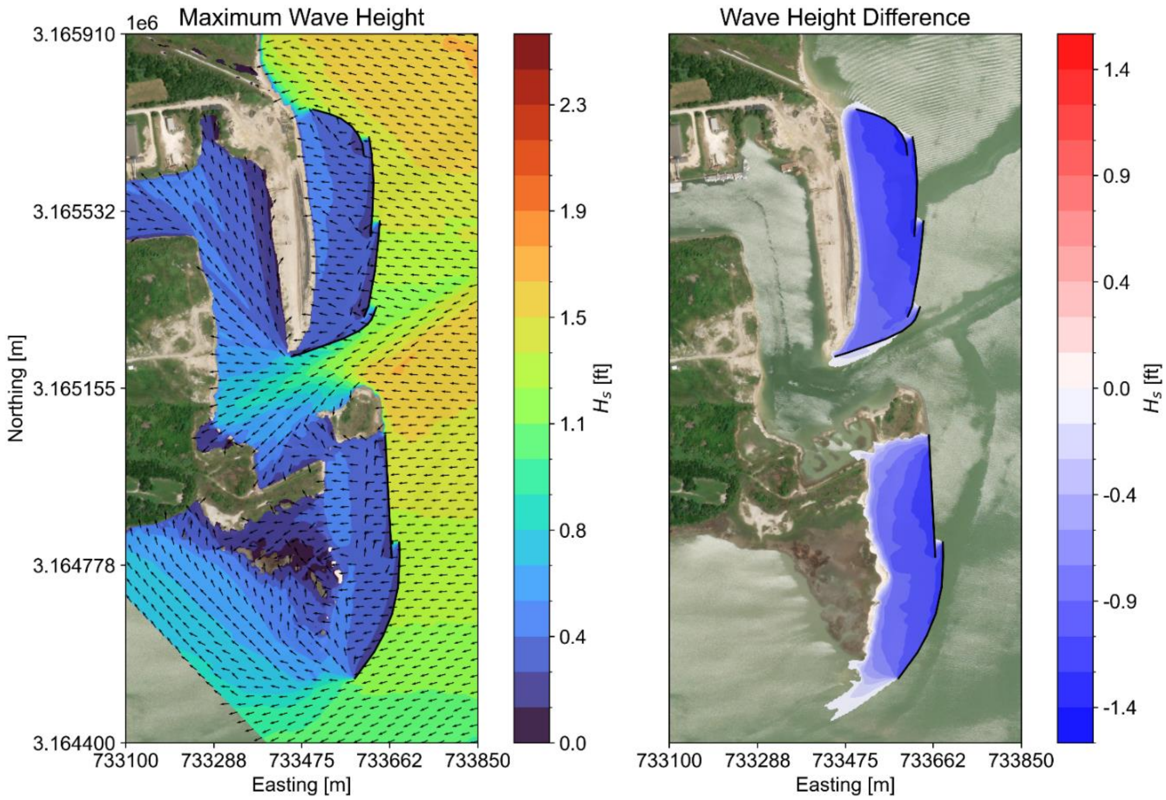

Figure 5.13: Simulated wave height and difference with existing conditions for Alternative 4A with the 1-year return period conditions from the east-southeast (105°).

5.6.2 Alternative 4B Erosion Protection

Alternative 4B, with a +6 ft NAVD88 crest elevation along the northern harbor peninsula, was modeled and analyzed using the methods discussed in Section 5.3. The effect of raising the crest elevation is to enhance wave attenuation during high-water events due to the structure's larger freeboard. Figure 5.14 plots the cross-shore profile for the initial bed level (black dashed line) prior to the 25-year event and the bed level after the 25-year event for the alternative (blue line) and existing conditions (green line).

The volume of material above MHHW after the simulation was calculated for existing conditions and Alternative 4B to determine erosion (ft^3/ft) for a 25-year event. The erosion rate for Alternative 4B is 0 ft^3/ft, a 100% reduction from existing conditions and approximately 10% more reduction compared to Alternatives 1 and 2 in Table 5.3. Although linear erosion is apparent (Figure 5.14), 100% of the eroded volume remained above the limits of calculation (MHHW). Further, the maximum linear erosion for Alternative 4B was 2.5 ft (a 50% reduction compared to existing conditions) with an average linear erosion of 0.6 ft (81% reduction). Lastly, results from Alternative 4B show that lifting the structure an addition 1 ft can improve protection against the average linear erosion by approximately 30%. Note that these erosion rates do not consider the effects of a vegetated shoreline or a marsh creation cell. Therefore, the simulated erosion reported here can be considered conservative.

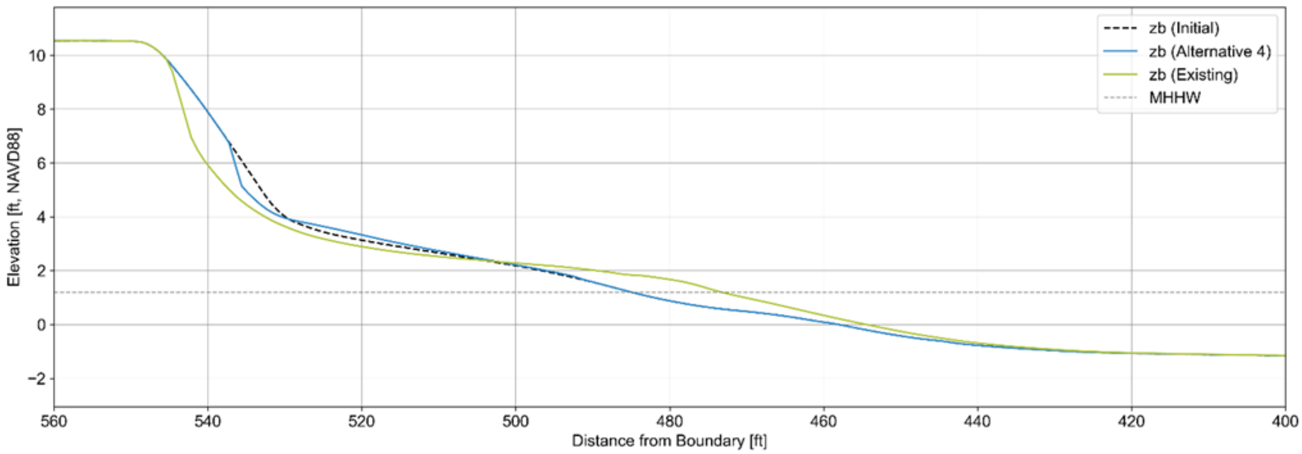

Figure 5.14: XBeach profile model results for Alternative 4B.

5.6.3 Alternative 4A and 4B Cost Adjustment

The cost of Alternative 4A is approximately $10.4 million (Table 5.9) and the cost of Alternative 4B is approximately $11.65 million (Table 5.10) while the cost of Alternative 2 is approximately $9.4 million (Table 5.5). The increased cost for Alternative 4A and 4B is due to the increased crest elevation for the northern rock breakwater and/or increased rock breakwater lengths from extending the gap protection overlap. The cost estimate breakdown for Alternative 4A and 4B is shown in Table 5.9 and Table 5.10.

Table 5.9: Alternative 4A – 5ft Crest Elevation Rock Breakwater

Item Description	Quantity	Unit	Unit Cost	Total
Mobilization and Demobilization	1	LS	$784,380.00	$784,380.00
Construction Surveying	5	EA	$20,000.00	$100,000.00
Environmental Protection	1	LS	$54,530.00	$54,530.00
Rock Breakwaters - Stone	34,928	TON	$176.00	$6,147,328.00
Rock Breakwaters - Geotextile	17,723	SY	$14.00	$248,122.00
Inner Harbor Reef Breakwater - Units	300	LF	$850.00	$255,000.00
Inner Harbor Reef Breakwater - Bedding Stone	223	TON	$99.00	$22,077.00
Inner Harbor Reef Breakwater - Geotextile	289	SY	$14.00	$4,046.00
ACBM	13,942	SF	$23.00	$320,666.00
ACBM - Geotextile	1,550	SY	$14.00	$21,700.00
ACBM - Bedding Stone	403	TON	$99.00	$39,897.00
Vegetated Slope - Soil Bags	19,539	SF	$32.00	$625,248.00
Vegetated Slope - Planting	0.5	AC	$10,209.00	$5,104.50
			Contingency (20%)	$1,725,619.70
			Total	$10,353,718.20

Table 5.10: Alternative 4B – 6ft Crest Elevation North Rock Breakwater, 5ft Crest Elevation South Rock Breakwater, and no vegetated slope protection along northern harbor shoreline.

Item Description	Quantity	Unit	Unit Cost	Total
Mobilization and Demobilization	1	LS	$818,480.00	$818,480.00
Construction Surveying	5	EA	$20,000.00	$100,000.00
Environmental Protection	1	LS	$56,900.00	$56,900.00
Rock Breakwaters - Stone	40,330	TON	$176.00	$7,098,080.00
Rock Breakwaters - Geotextile	19,028	SY	$14.00	$266,392.00
Inner Harbor Reef Breakwater - Units	300	LF	$850.00	$255,000.00
Inner Harbor Reef Breakwater - Bedding Stone	223	TON	$99.00	$22,077.00
Inner Harbor Reef Breakwater - Geotextile	289	SY	$14.00	$4,046.00
ACBM	13,942	SF	$23.00	$320,666.00
ACBM - Geotextile	1,550	SY	$14.00	$21,700.00
ACBM - Bedding Stone	403	TON	$99.00	$39,897.00
Vegetated Slope – Soil Bags	0	SF	$32.00	$0.00
Vegetated Slope – Planting	0	AC	$10,209.00	$0.00
			Contingency (20%)	$1,800,647.60
			Total	$10,803,885.60

If construction funds are limited, another option for erosion control along the northern harbor peninsula could be to install a combination of rock revetment (+1 ft to +6 ft NAVD88) and vegetation sandbags (+6 ft to +11.5 ft NAVD88) on the shoreline in place of an offshore breakwater. The cost for this option is summarized in Table 5.11 and would nearly cut the cost of construction in half. The disadvantage is the loss of ecological benefit provided by the rock breakwater with a successful marsh creation along the northern project shoreline, which was a stated project goal. While marsh could be created on the northern project shoreline, without breakwater protection it is expected to erode. However, marsh creation and the beneficial use of dredge material would still be possible along the southern project shoreline.

Table 5.11: Northern harbor peninsula rock revetment, 5ft Crest Elevation South Rock Breakwater option.

Item Description	Quantity	Unit	Unit Cost	Total
Mobilization and Demobilization	1	LS	$526,900.00	$526,900.00
Construction Surveying	5	EA	$20,000.00	$100,000.00
Environmental Protection	1	LS	$36,630.00	$36,630.00
Rock Breakwaters - Stone	14,638	TON	$176.00	$2,576,288.00
Rock Breakwaters - Geotextile	7,428	SY	$14.00	$103,992.00
Inner Harbor Reef Breakwater - Units	300	LF	$850.00	$255,000.00
Inner Harbor Reef Breakwater - Bedding Stone	223	TON	$99.00	$22,077.00
Inner Harbor Reef Breakwater - Geotextile	289	SY	$14.00	$4,046.00
ACBM	13,942	SF	$23.00	$320,666.00
ACBM - Geotextile	1,550	SY	$14.00	$21,700.00
ACBM - Bedding Stone	403	TON	$99.00	$39,897.00
Vegetated Slope - Soil Bags	10,419	SF	$32.00	$333,408.00
Vegetated Slope - Planting	0.3	AC	$10,209.00	$3,062.70
Revetment - Stone	7,865	TON	$176.00	$1,384,240.00
Revetment - Geotextile	4,856	SY	$14.00	$67,984.00
			Contingency (20%)	$868,733.34
			Total	$5,212,400.04

5.7 Recommended Alternative

The recommended alternative is Alternative 4A. This option, constructed with the vegetated slope along the northern harbor shoreline, is expected to perform similarly to Alternative 4B without vegetated slope protection and is over $500,000 cheaper.

5.8 Next Steps in Design

The following actions, at a minimum, are anticipated to be performed to advance the feasibility level design to a permit level design package:

- Layout refinements during preliminary design
- Refine breakwater toe design based on scour analysis.
- Conduct settlement analysis and slope stability analysis.
- Conduct breakwater foundation design based on geotechnical data.
- Finalize cost estimate based on available funding for permit level design.

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A. Appendix A

A.1 Alternative performance - Wave Attenuation

Table A.1 Wave height reduction and % reduction between Alternatives and Existing Conditions

Tr [year]	Alternative	Reach	Absolute Hs diff [ft]	% Difference
1	Alternative 1	Inner	-0.06	-7.8
1	Alternative 2	Inner	-0.07	-9.1
1	Alternative 3A	Inner	-0.05	-7.1
1	Alternative 3B	Inner	-0.06	-7.8
1	Alternative 1	North	-0.63	-46.0
1	Alternative 2	North	-0.62	-45.2
1	Alternative 3A	North	-0.63	-46.0
1	Alternative 3B	North	-0.63	-46.0
1	Alternative 1	South	-0.83	-61.2
1	Alternative 2	South	-0.87	-64.1
1	Alternative 3A	South	-0.53	-39.3
1	Alternative 3B	South	-0.71	-52.3
5	Alternative 1	Inner	-0.09	-8.7
5	Alternative 2	Inner	-0.10	-10.1
5	Alternative 3A	Inner	-0.07	-7.1
5	Alternative 3B	Inner	-0.09	-8.6
5	Alternative 1	North	-0.77	-42.8
5	Alternative 2	North	-0.75	-41.8
5	Alternative 3A	North	-0.77	-42.8
5	Alternative 3B	North	-0.77	-42.8
5	Alternative 1	South	-1.06	-58.6
5	Alternative 2	South	-1.12	-61.4
5	Alternative 3A	South	-0.47	-25.9
5	Alternative 3B	South	-0.80	-44.1
25	Alternative 1	Inner	-0.05	-2.1
25	Alternative 2	Inner	-0.05	-2.4
25	Alternative 3A	Inner	-0.04	-1.8
25	Alternative 3B	Inner	-0.05	-2.1
25	Alternative 1	North	-0.47	-18.1
25	Alternative 2	North	-0.49	-18.7
25	Alternative 3A	North	-0.47	-18.1
25	Alternative 3B	North	-0.47	-18.1
25	Alternative 1	South	-0.88	-31.1
25	Alternative 2	South	-0.92	-32.8
25	Alternative 3A	South	-0.19	-6.7
25	Alternative 3B	South	-0.52	-18.4
100	Alternative 1	Inner	-0.02	-0.5
100	Alternative 2	Inner	-0.02	-0.5
100	Alternative 3A	Inner	-0.01	-0.3
100	Alternative 3B	Inner	-0.02	-0.5
100	Alternative 1	North	-0.25	-5.7
100	Alternative 2	North	-0.26	-5.9
100	Alternative 3A	North	-0.25	-5.8
100	Alternative 3B	North	-0.25	-5.8
100	Alternative 1	South	-0.53	-11.5
100	Alternative 2	South	-0.55	-11.9
100	Alternative 3A	South	-0.15	-3.3
100	Alternative 3B	South	-0.31	-6.7

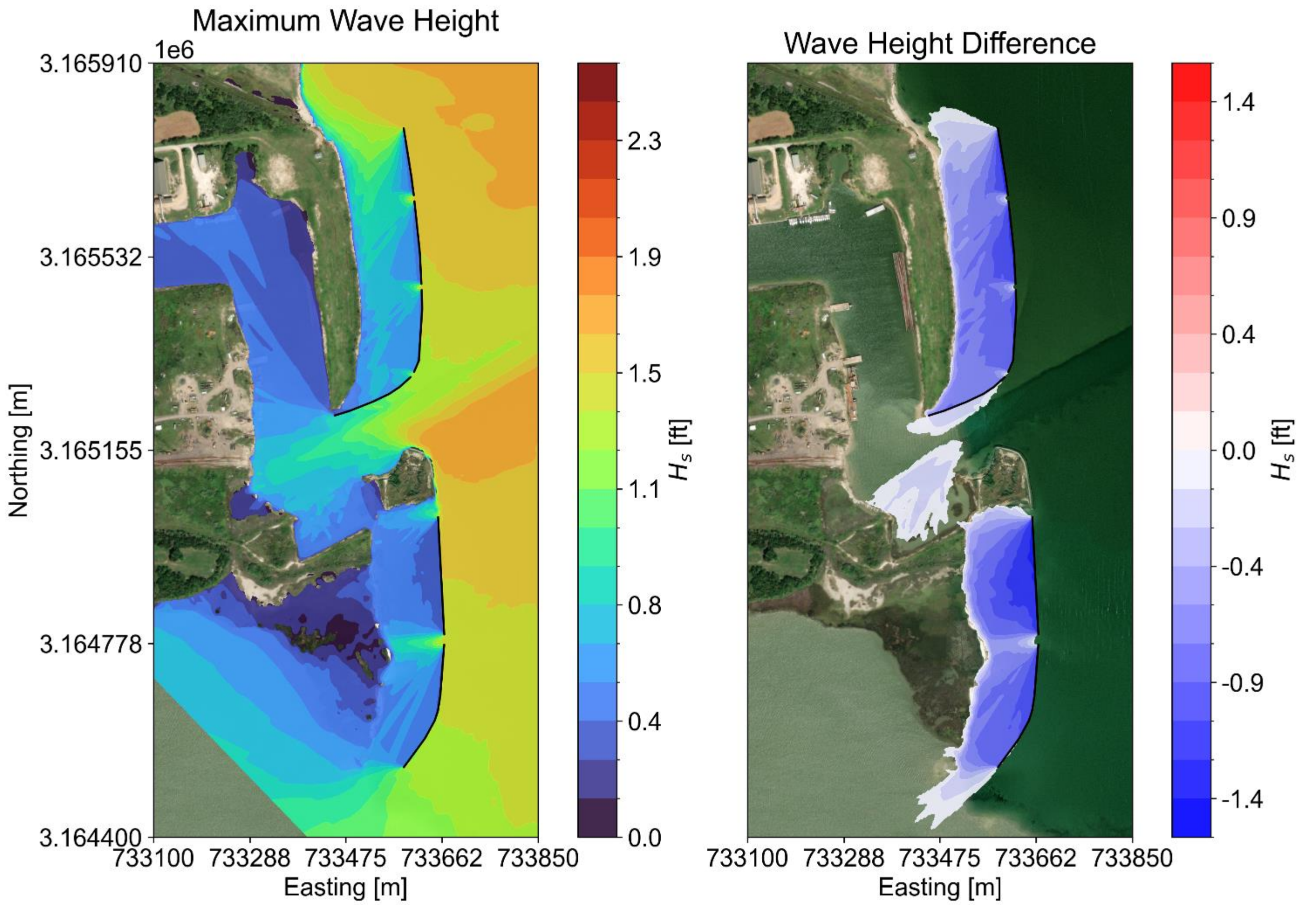

Figure A.1: Simulated wave height and difference with existing conditions for Alternative 1 with the 1-year return period conditions.

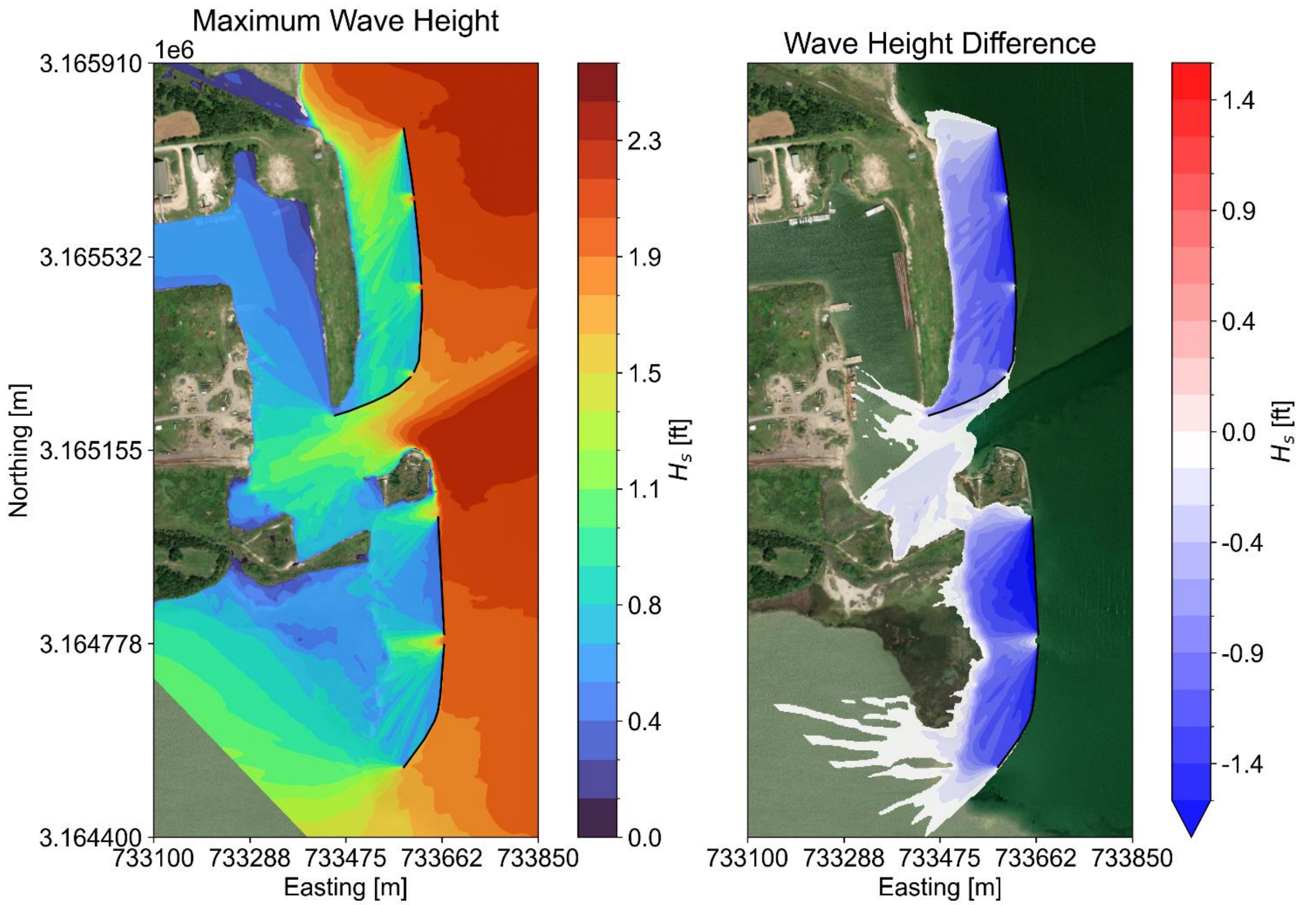

Figure A.2: Simulated wave height and difference with existing conditions for Alternative 1 with the 5-year return period conditions.

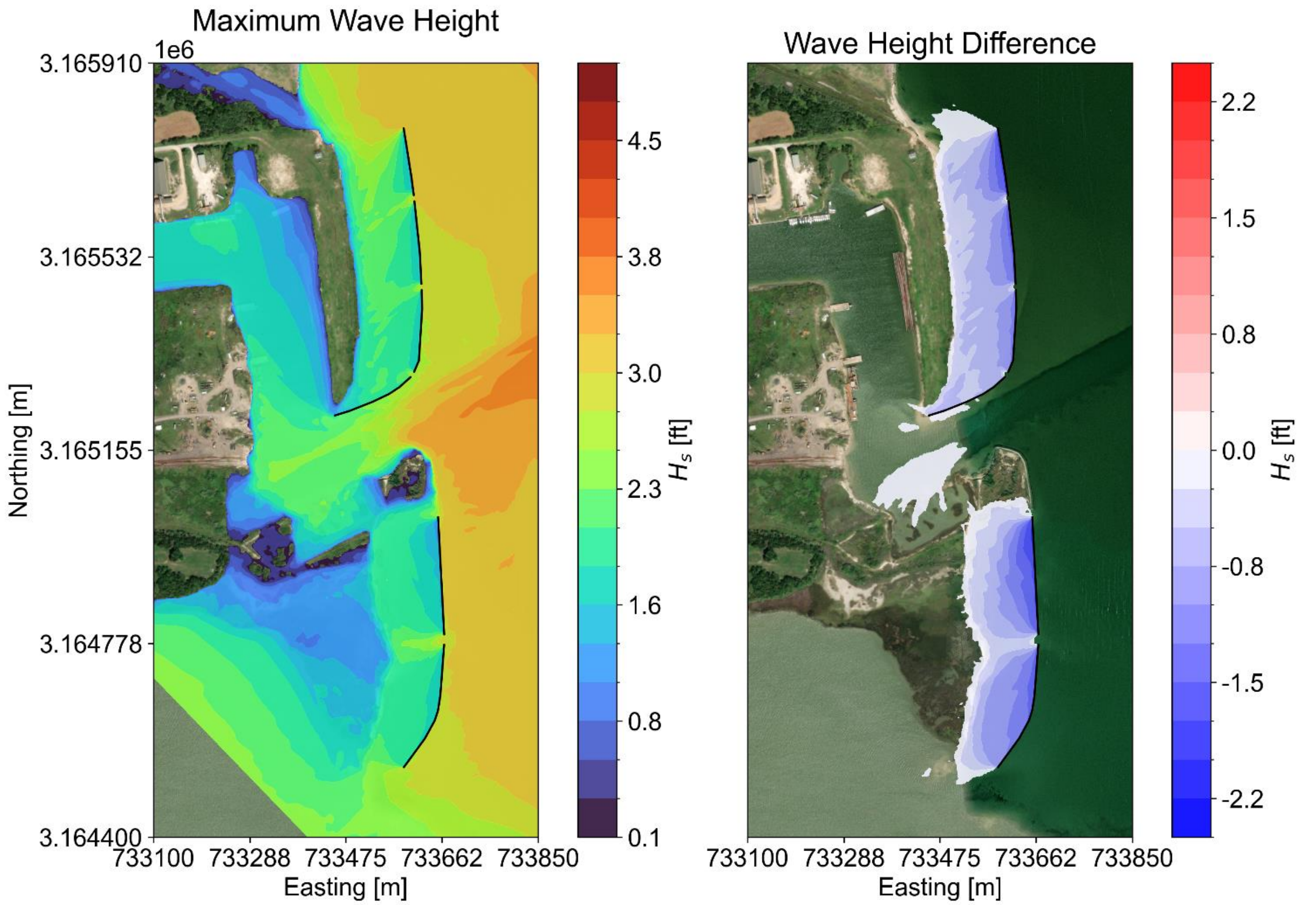

Figure A.3: Simulated wave height and difference with existing conditions for Alternative 1 with the 25-year return period conditions.

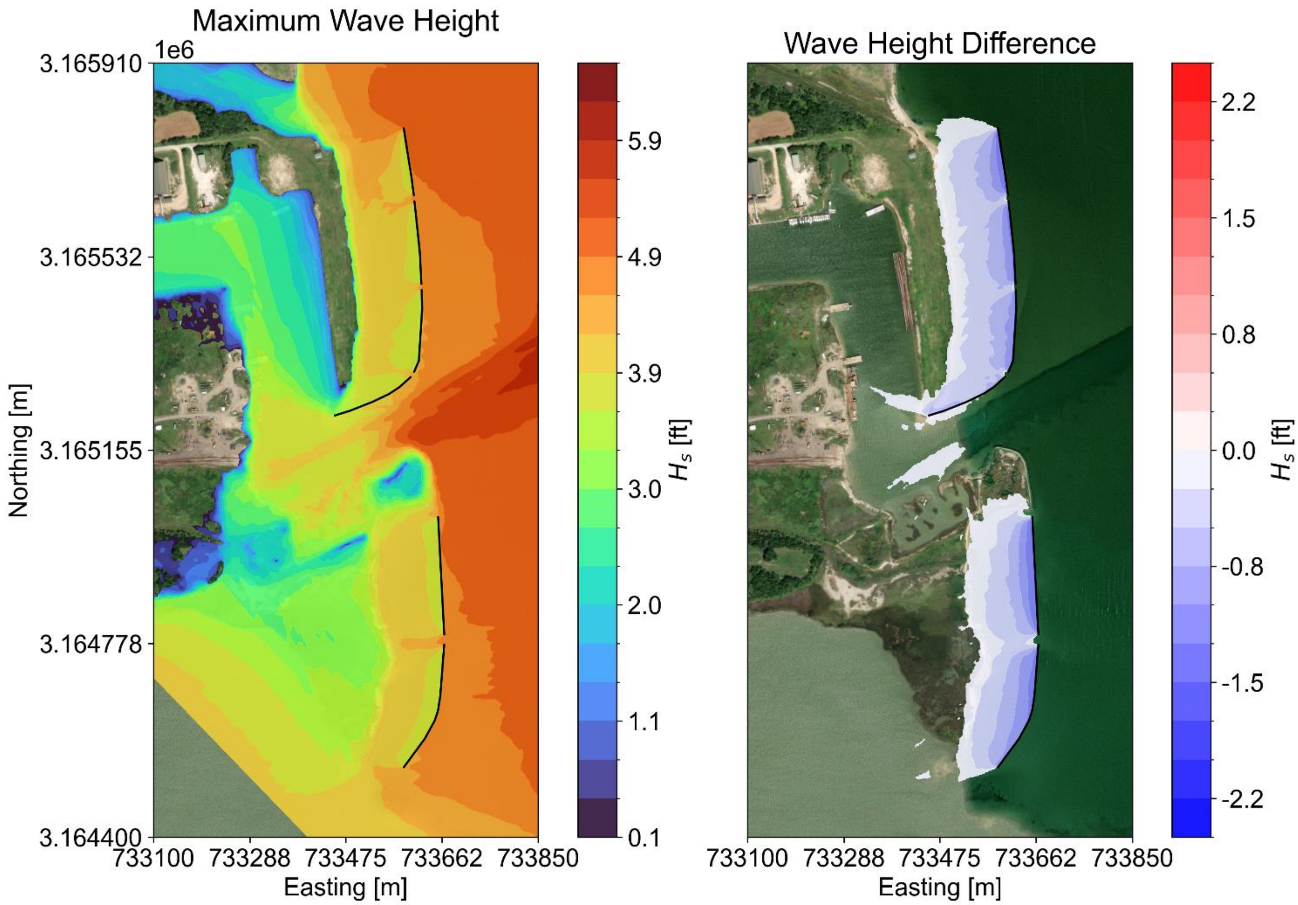

Figure A.4: Simulated wave height and difference with existing conditions for Alternative 1 with the 100-year return period conditions.

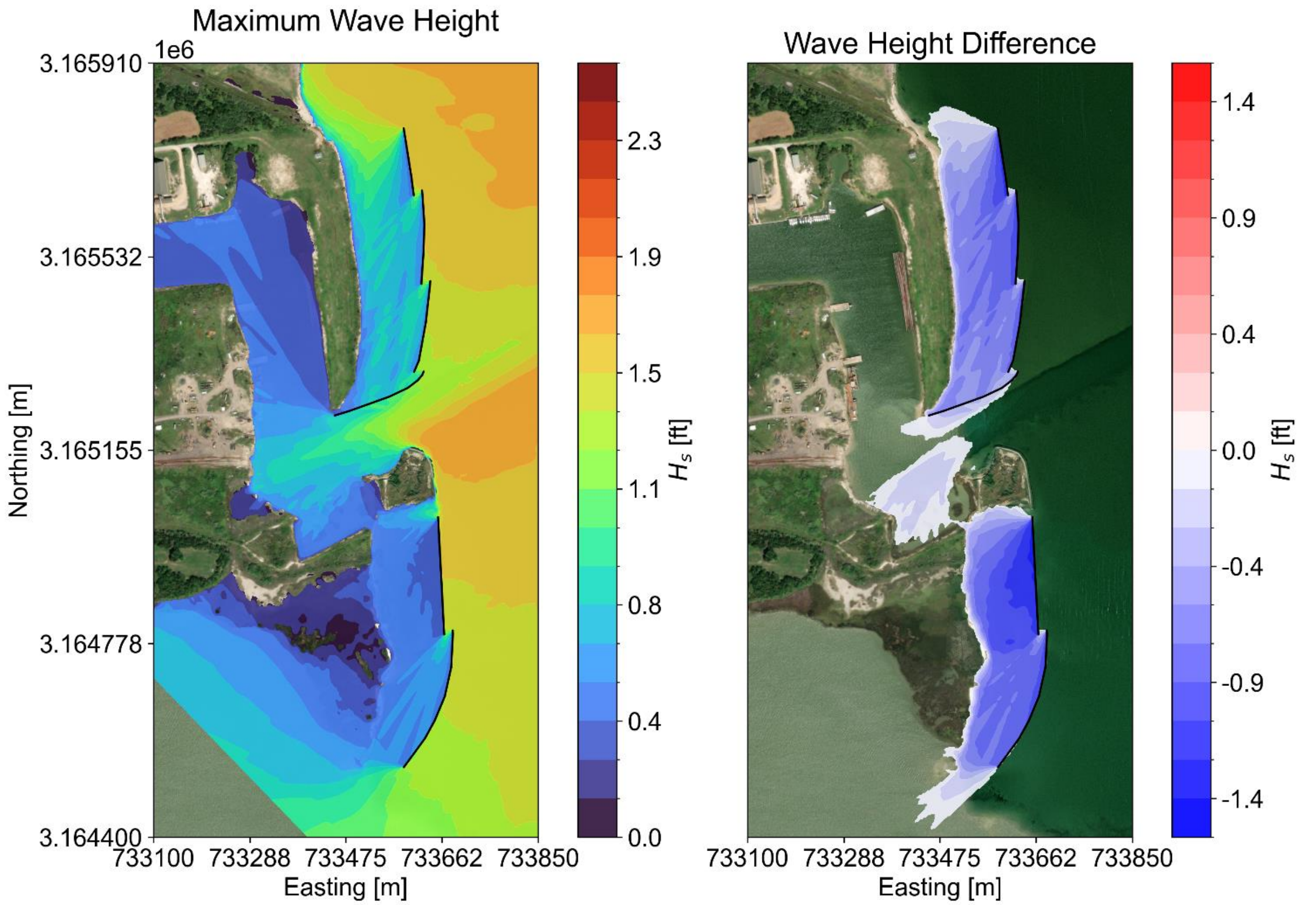

Figure A.5: Simulated wave height and difference with existing conditions for Alternative 2 with the 1-year return period conditions.

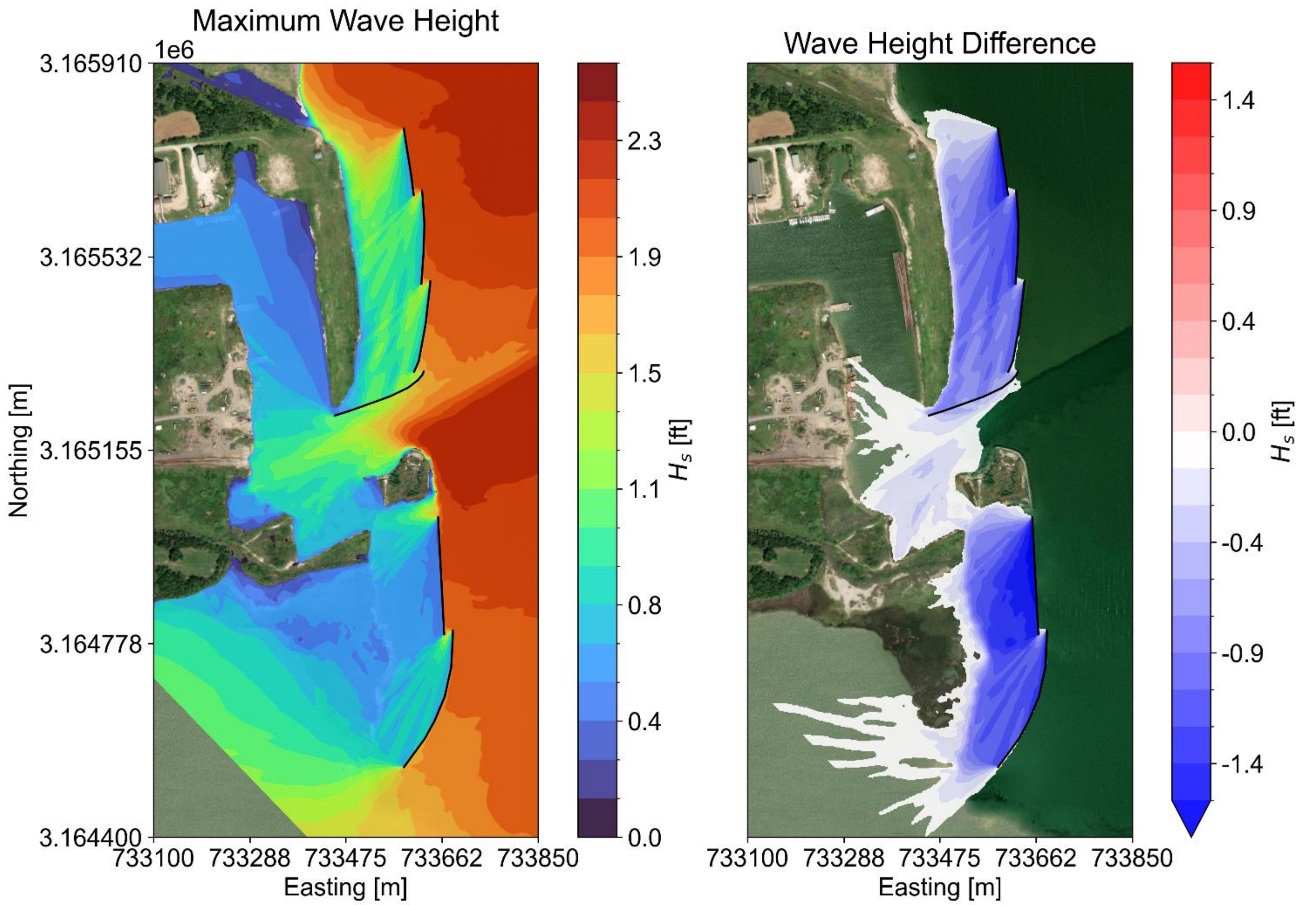

Figure A.6: Simulated wave height and difference with existing conditions for Alternative 2 with the 5-year return period conditions.

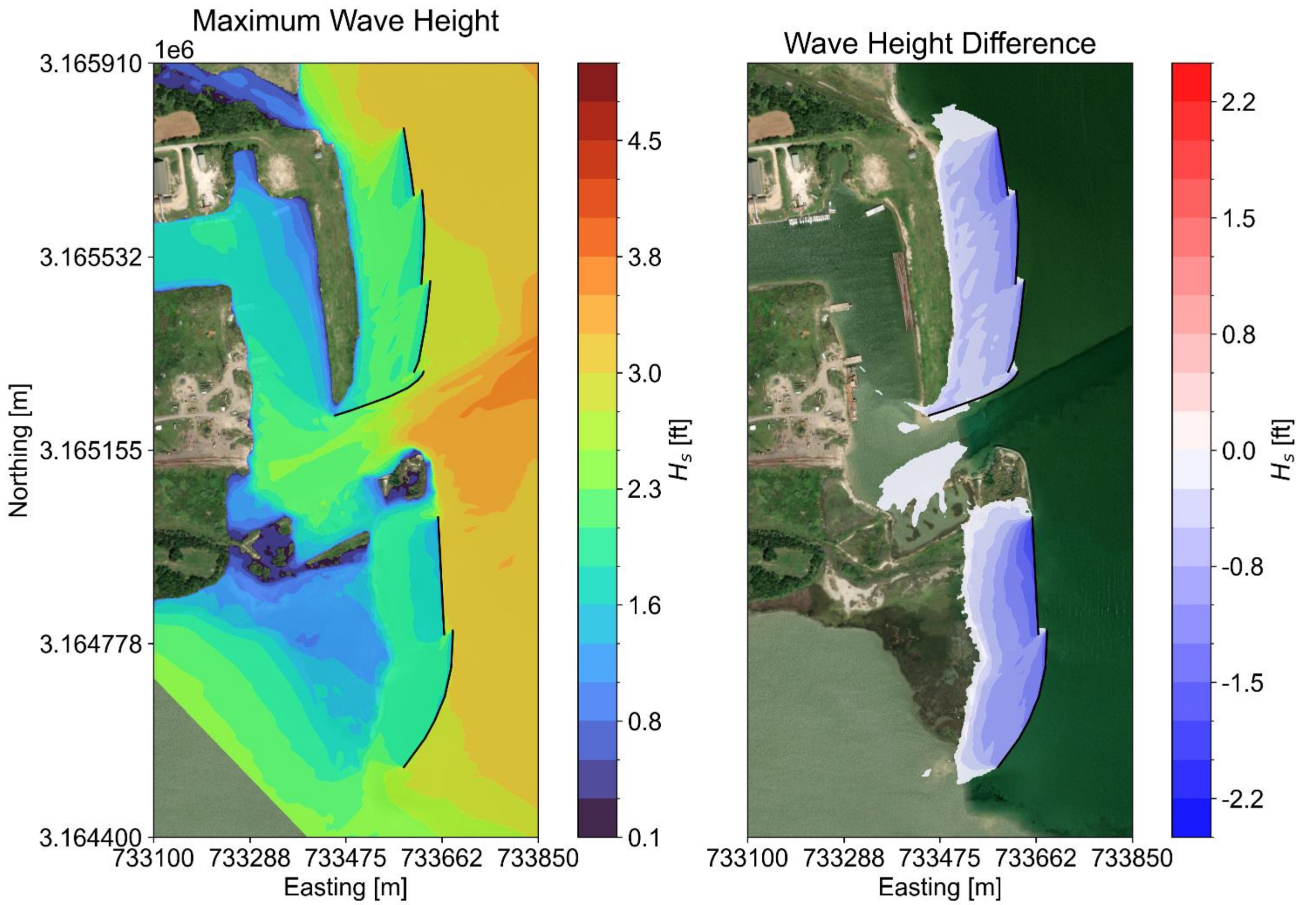

Figure A.7: Simulated wave height and difference with existing conditions for Alternative 2 with the 25-year return period conditions.

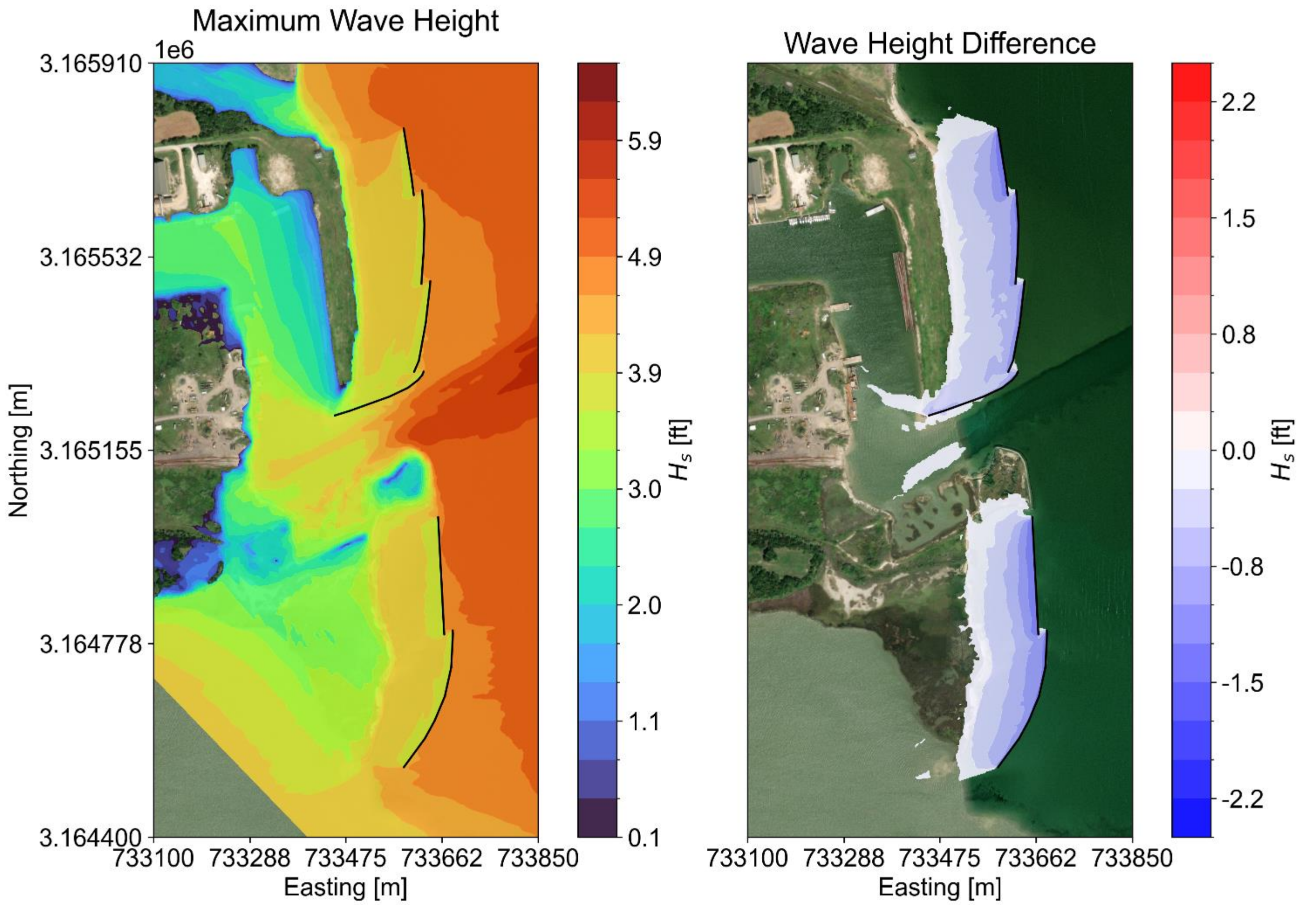

Figure A.8: Simulated wave height and difference with existing conditions for Alternative 2 with the 100-year return period conditions.

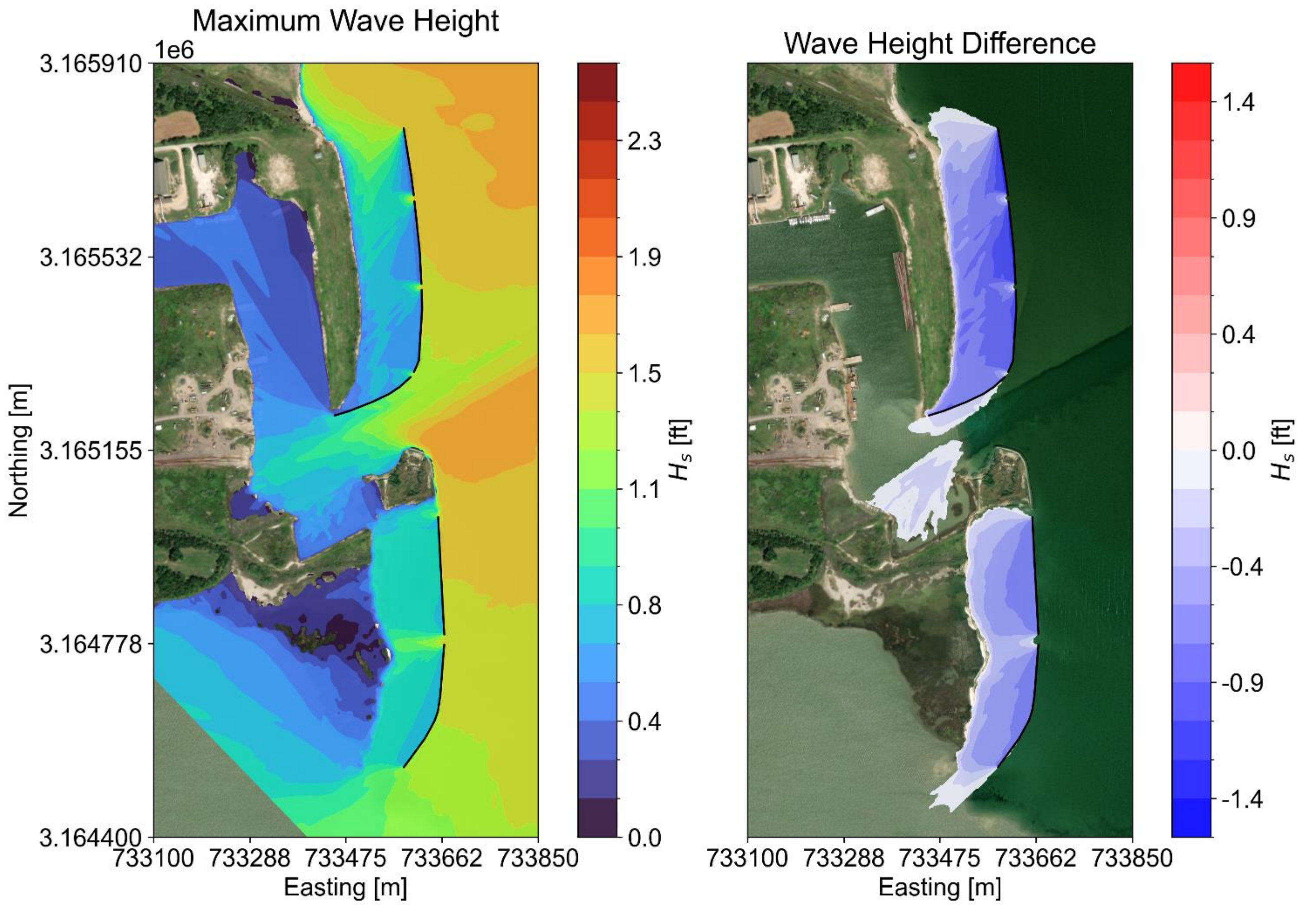

Figure A.9: Simulated wave height and difference with existing conditions for Alternative 3A with the 1-year return period conditions.

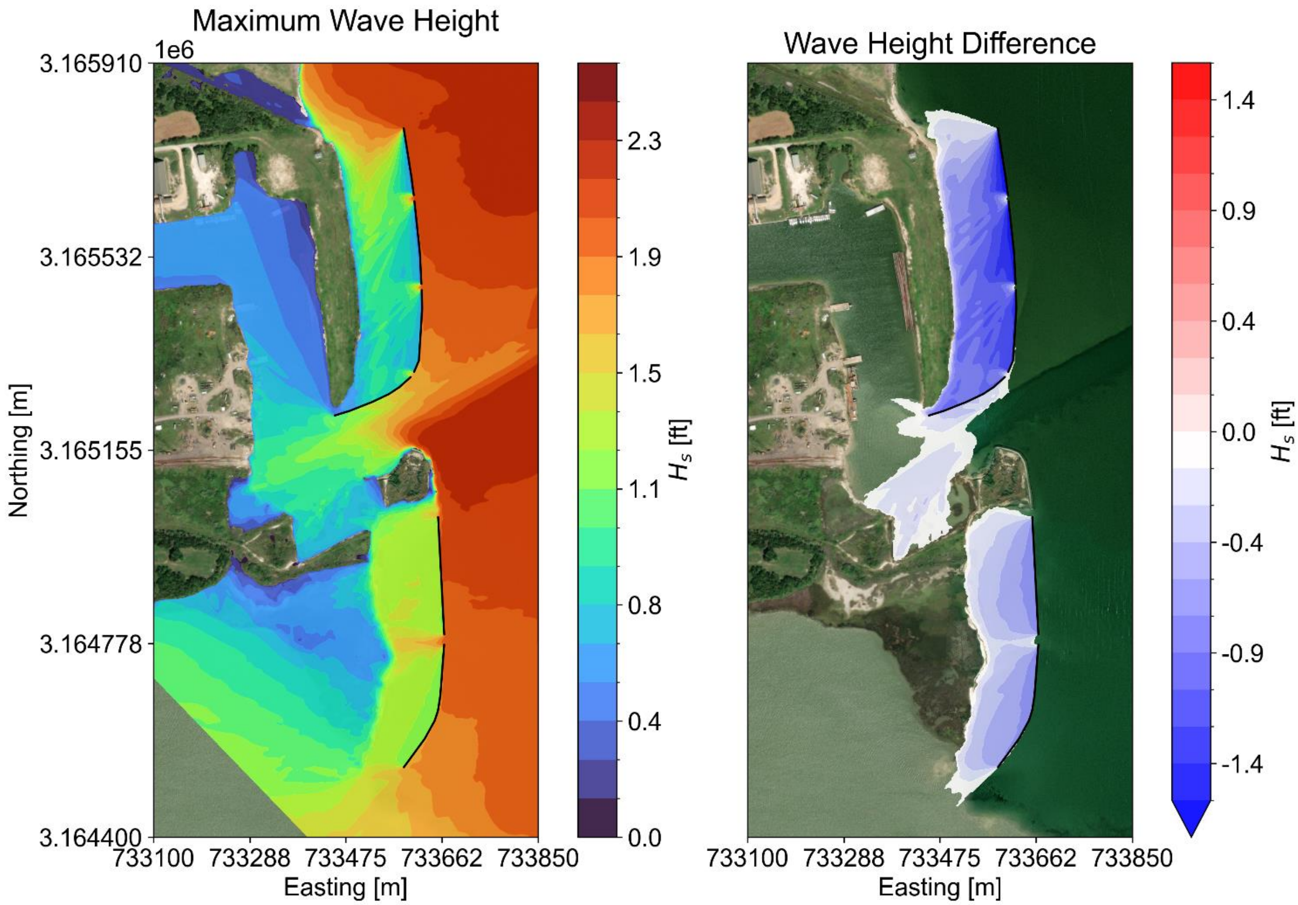

Figure A.10: Simulated wave height and difference with existing conditions for Alternative 3A with the 5-year return period conditions.

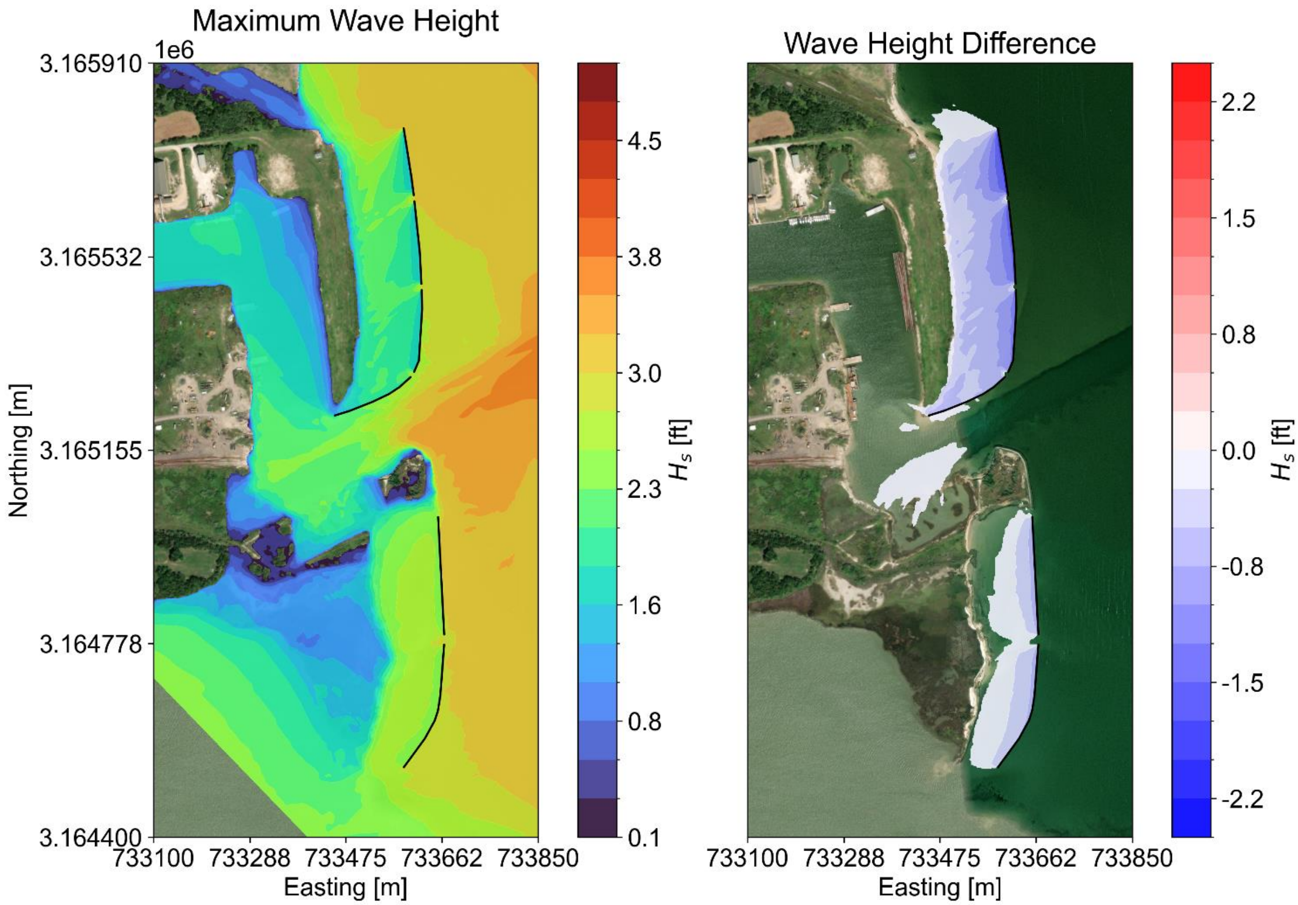

Figure A.11: Simulated wave height and difference with existing conditions for Alternative 3A with the 25-year return period conditions.

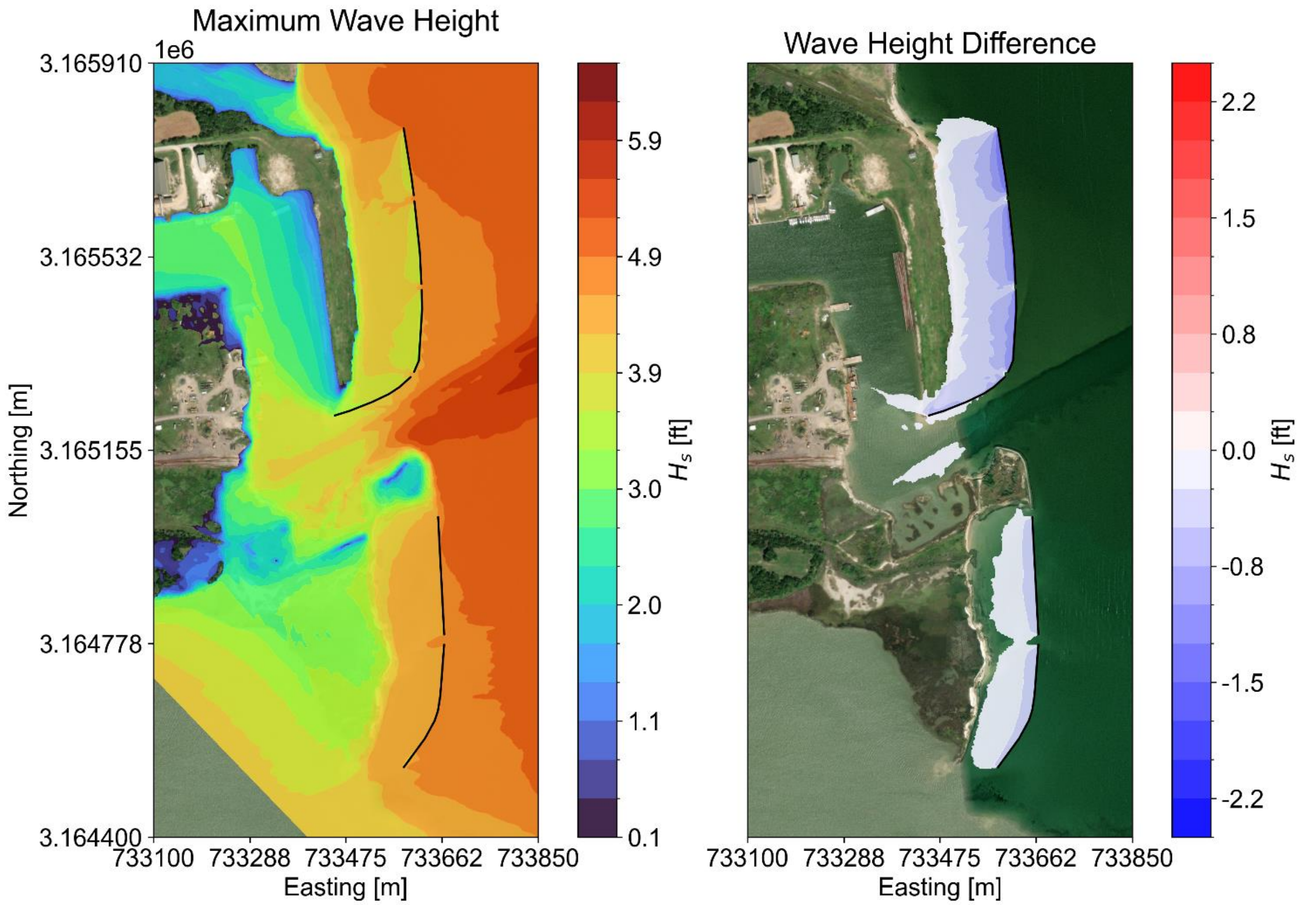

Figure A.12: Simulated wave height and difference with existing conditions for Alternative 3A with the 100-year return period conditions.

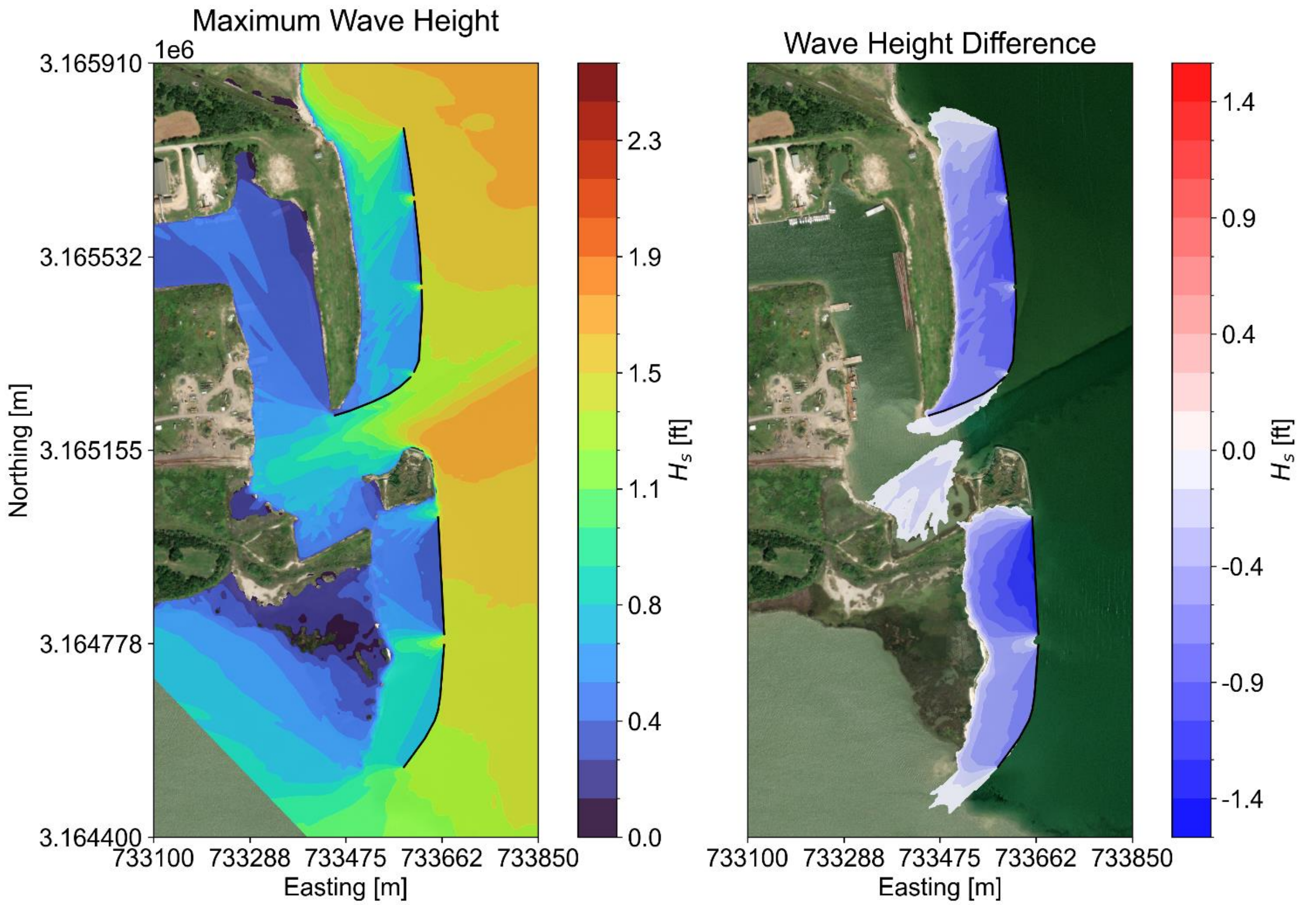

Figure A.13: Simulated wave height and difference with existing conditions for Alternative 3B with the 1-year return period conditions.

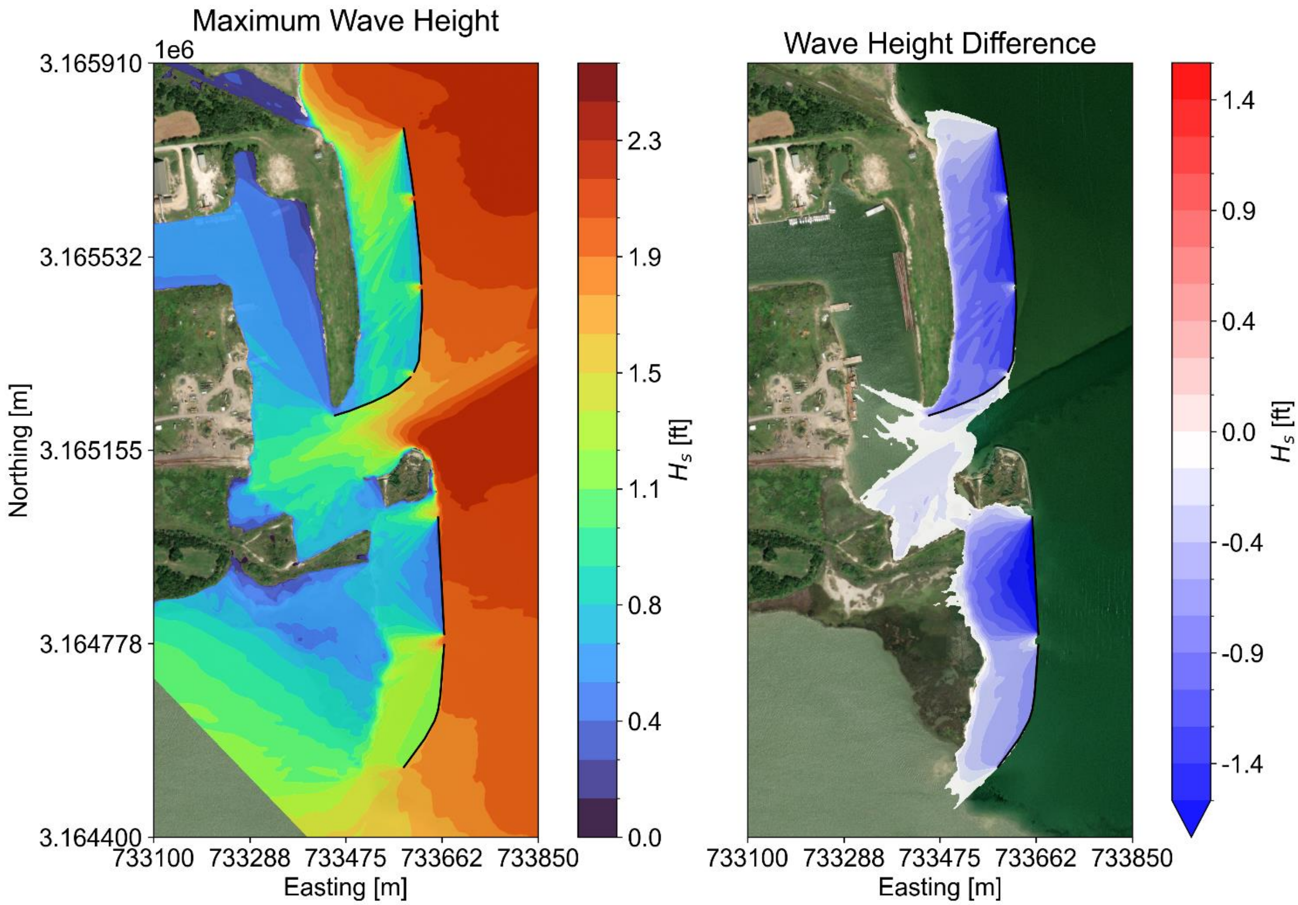

Figure A.14: Simulated wave height and difference with existing conditions for Alternative 3B with the 5-year return period conditions.

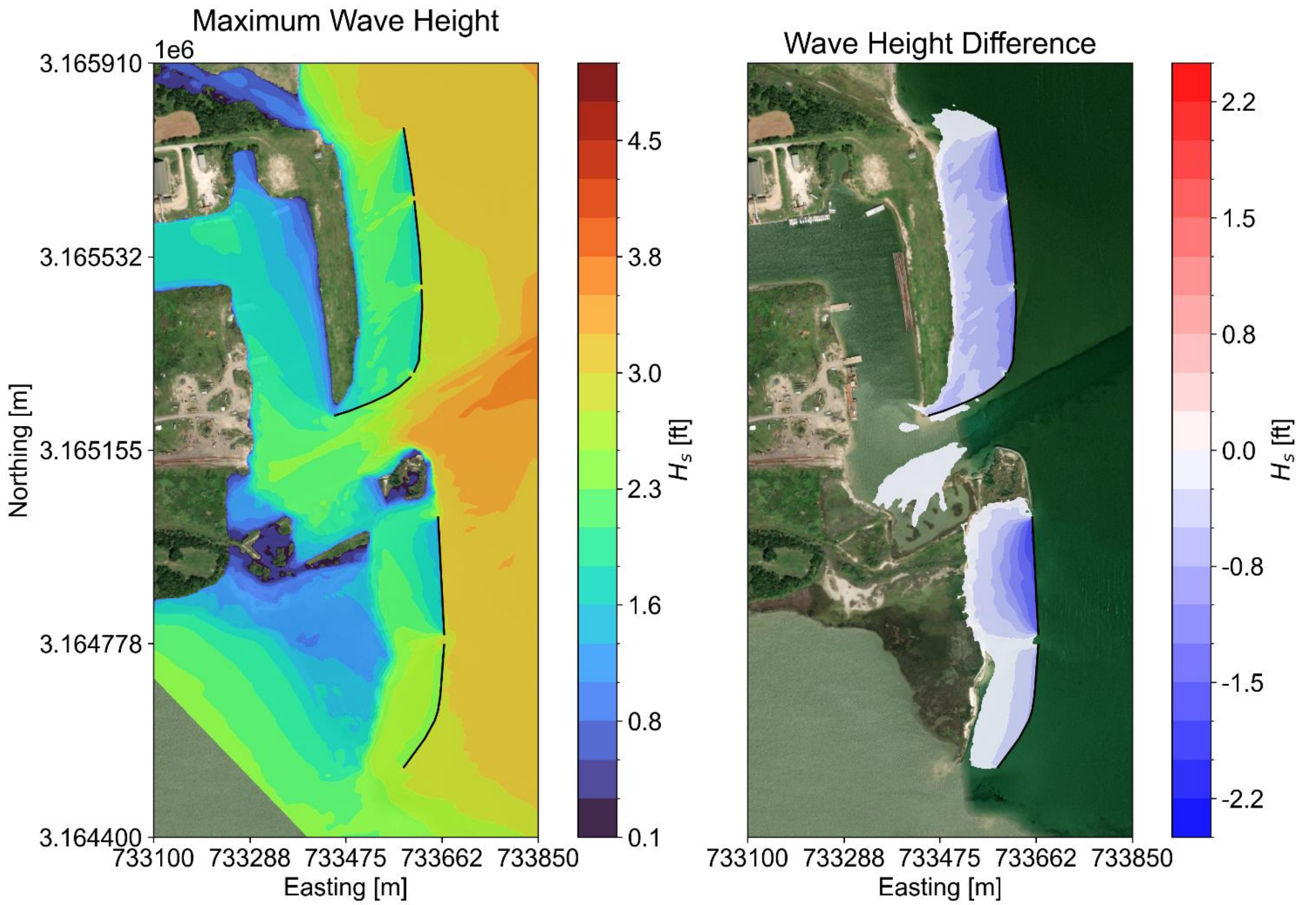

Figure A.15: Simulated wave height and difference with existing conditions for Alternative 3B with the 25-year return period conditions.

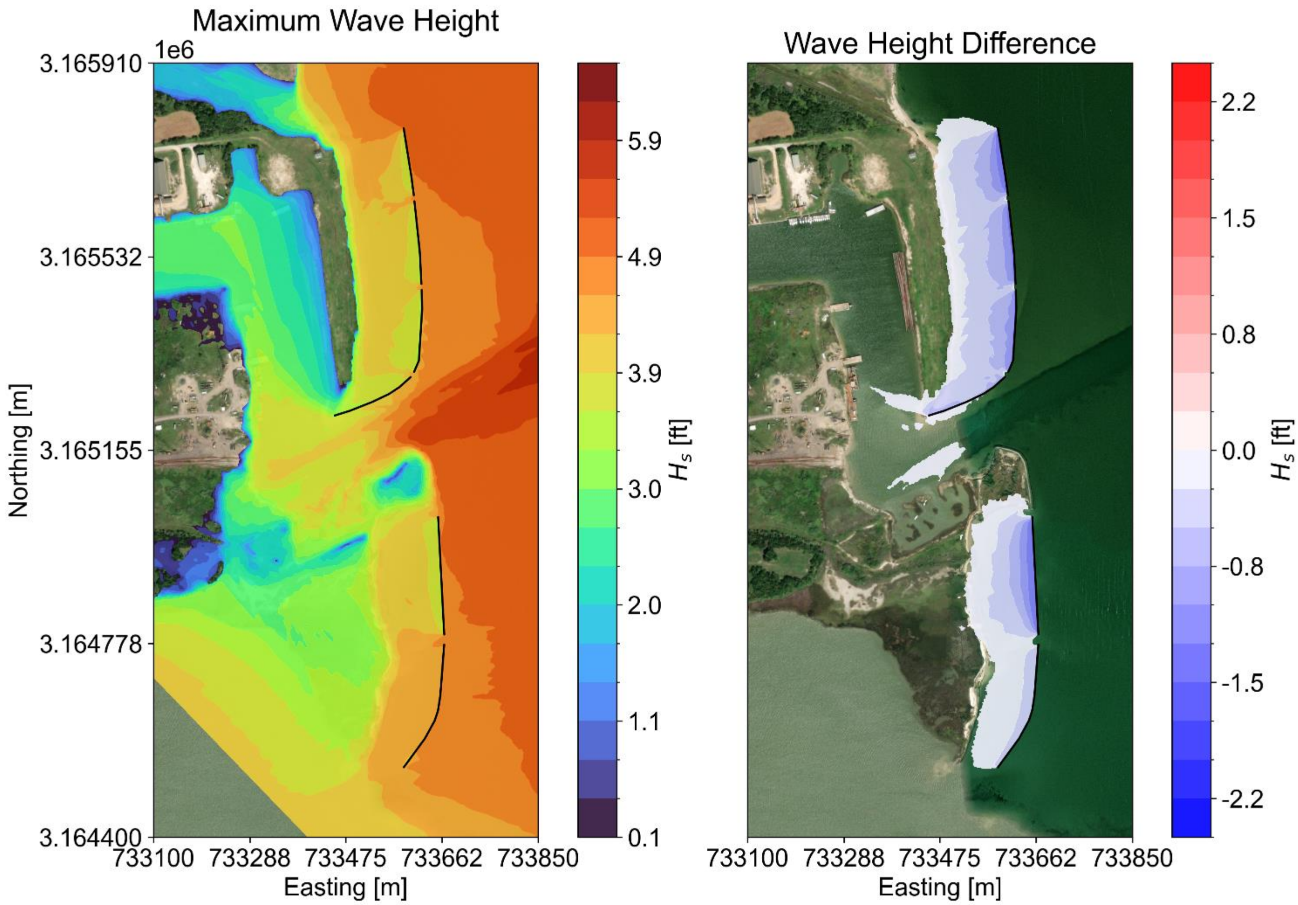

Figure A.16: Simulated wave height and difference with existing conditions for Alternative 3B with the 100-year return period conditions.

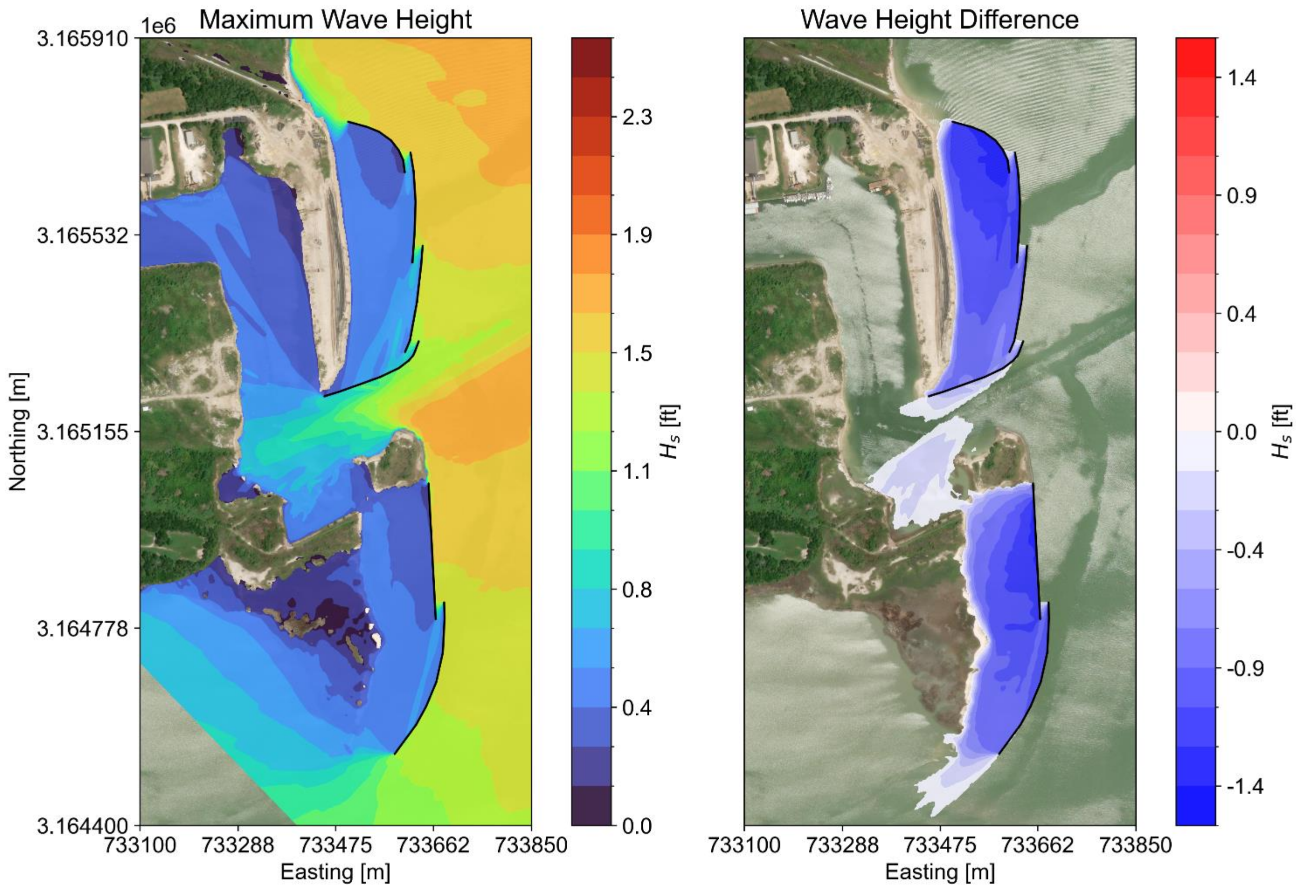

Figure A.17: Simulated wave height and difference with existing conditions for Alternative 4 with the 1-year return period conditions.

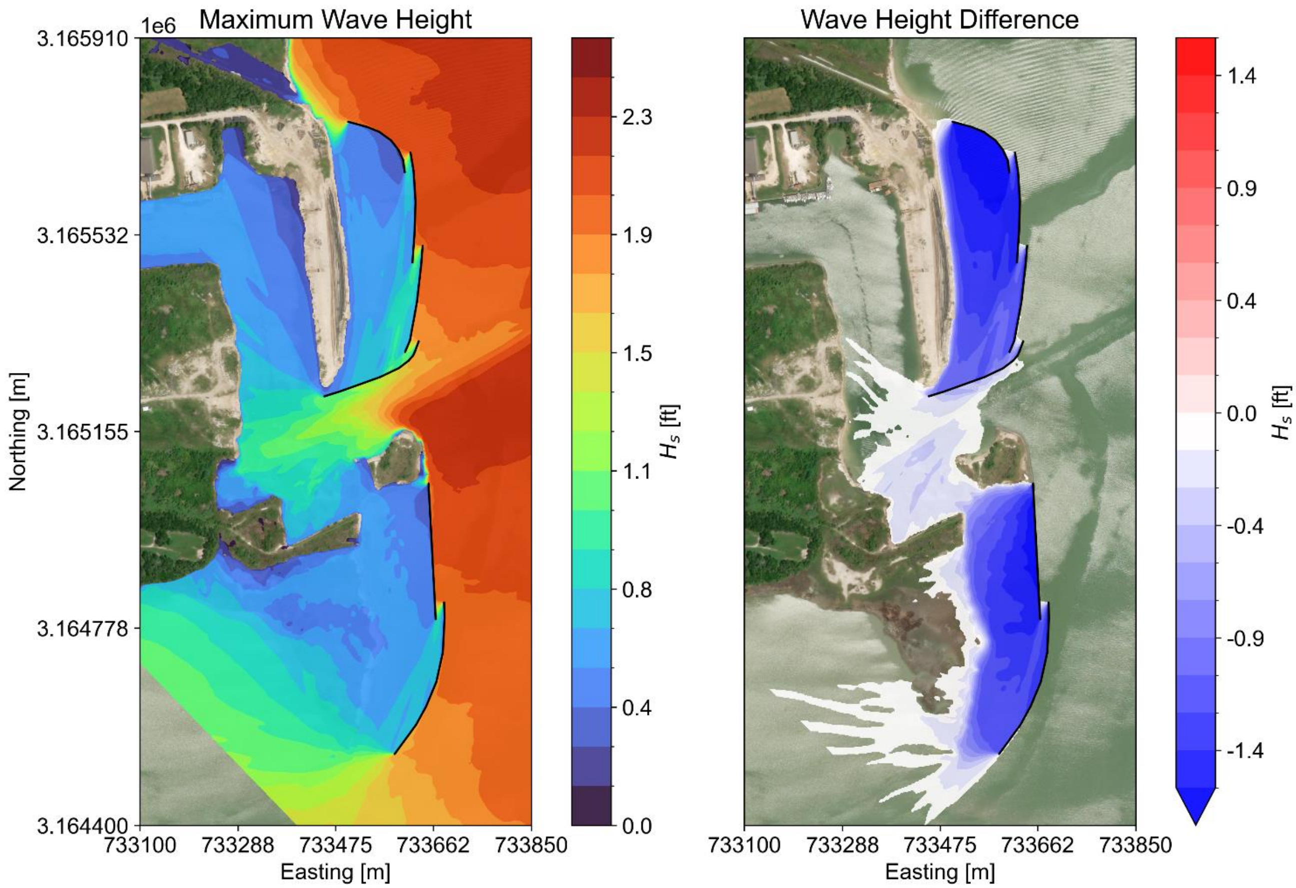

Figure A.18: Simulated wave height and difference with existing conditions for Alternative 4 with the 5-year return period conditions.

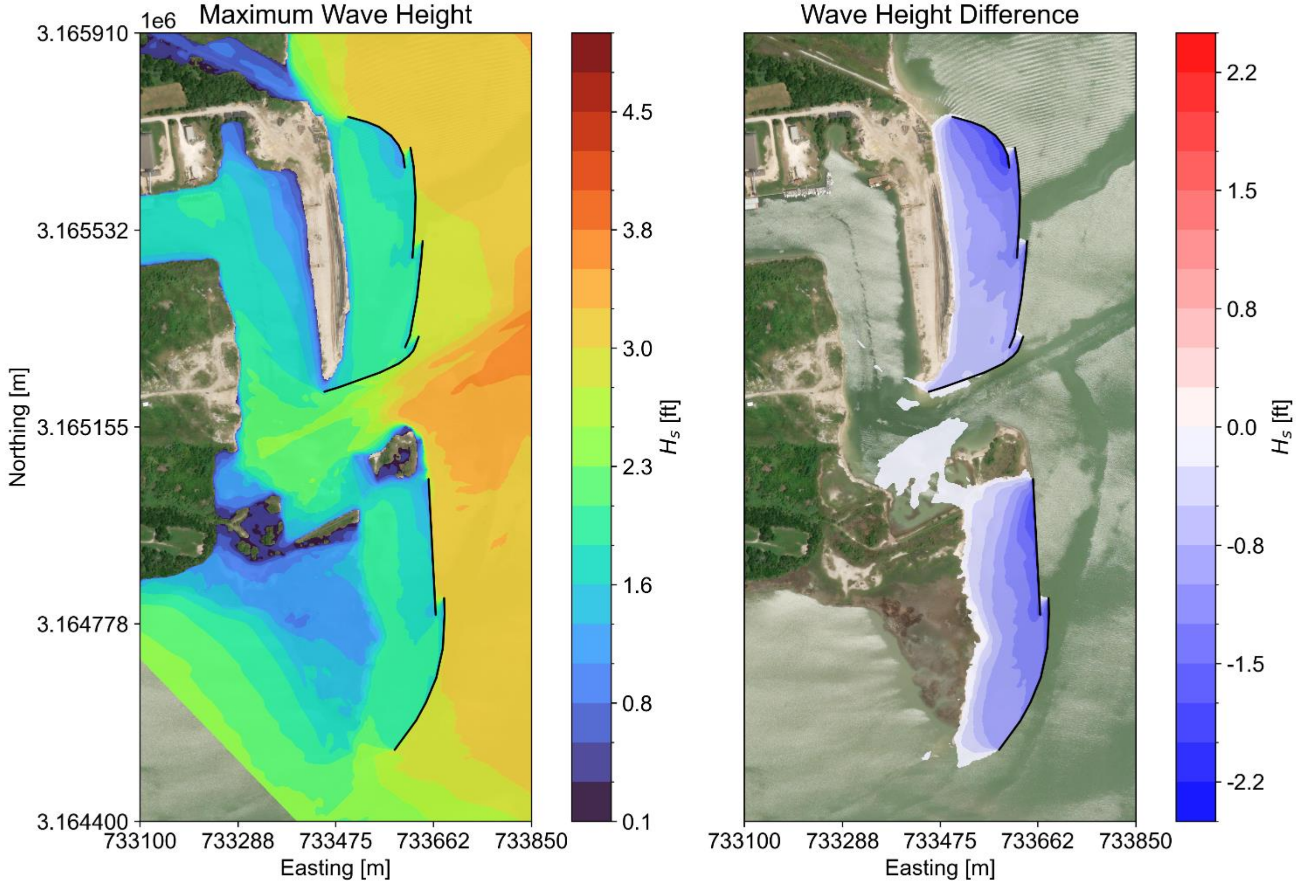

Figure A.19: Simulated wave height and difference with existing conditions for Alternative 4 with the 25-year return period conditions.

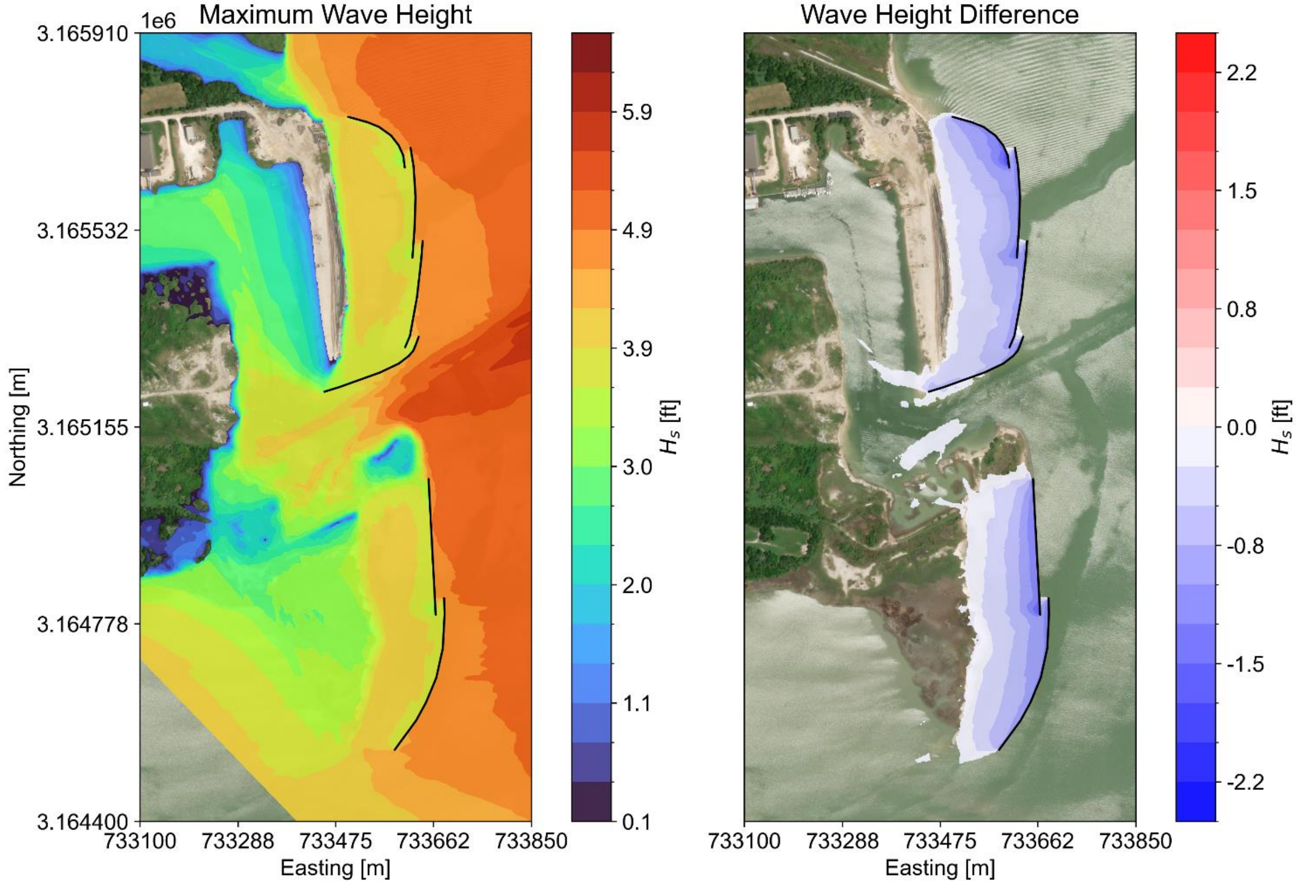

Figure A.20: Simulated wave height and difference with existing conditions for Alternative4 with the 100-year return period conditions.