Coast & Harbor Engineering
A Division of Hatch Mott MacDonald

Cline's Point Marina Ship Wave Analysis

Draft Technical Report

April 13, 2015

Submitted To:
Port of Corpus Christi Authority

PORTCORPUSCHRISTI

Prepared for:

Port of Corpus Christi Authority

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Respectfully Submitted,

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April 13, 2015

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EXECUTIVE SUMMARY

Cline's Point Marina, located in Port Aransas, TX, has been experiencing detrimental conditions within the marina during the passage of deep-draft ship traffic. The wave activity is primarily generated by pressure field effects from large, laden outbound vessels in Corpus Christi Ship Channel (CCSC). The basin was originally protected by an approximately 180 ft. long breakwater constructed in 1976 on the west side of the entrance which by 1980 has deteriorated and partially failed to 120 ft long breakwater and continued deteriorating over time, reaching its approximate 60 ft long current condition by 2003. The deterioration of this breakwater has reportedly resulted in enhanced penetration of deep-draft vessel wave activity into the marina.

The work performed as part of this study consists of evaluating the mechanisms by which waves are generated and enter the harbor, quantifying the level of protection afforded by past and present entrance breakwater configuration(s) and developing conceptual alternatives for improving conditions inside the marina. This report presents the data collected for the study, ship hydrodynamic modeling results used to evaluate the wave penetration mechanism inside the marina, development and analysis of conceptual alternatives, as well as conceptual-level cost estimates.

Deep-draft vessel-induced surge analysis was performed to evaluate water level fluctuations and surge-induced current velocities inside the marina generated by passing vessels. The Vessel Hydrodynamics Longwave Unsteady (VH-LU) modeling system (Fenical et al 2006) is a comprehensive hydrodynamic modeling system that calculates water level and current velocity fluctuations generated by moving deep-draft vessels. It was concluded from the modeling of present-day breakwater (existing) conditions that when tankers are passing at higher speeds, the pressure field passing the entrance to the marina introduces both long-period water level oscillations and shorter-period wave components that break and potentially cause impacts to boats and interior structures.

Alternatives were developed and analyzed for reducing the penetration of both the long-period water level oscillations and shorter-period breaking waves into the marina. The interior of the marina acts as a flat beach which effectively allows the formation and propagation of breaking (bore) waves, therefore deepening of the marina may help in reducing the formation of these waves. Also, the wider entrance configuration now present following degradation of the length of the west breakwater allows more short-period wave energy to propagate inside the marina. Therefore restoration of the breakwater or other structural modification of the marina entrance and/or deepening within the marina may help in reducing the penetration of these waves into the marina or the impacts of these waves after they enter the marina.

The various alternatives that were developed in discussion with PCCA were:

- Alternative 0: Existing condition, the 60 ft. breakwater
- Alternative 1: Add 120 ft. of new breakwater to and at same orientation as existing breakwater.

- Alternative 2: Add 60 ft. of new breakwater to and at same orientation as existing breakwater.
- Alternative 3: Add 74 ft. of new breakwater to and angled from the existing breakwater.
- Alternative 3A: 66 ft. of new breakwater angled from the eastern bulkhead opposite the existing breakwater.
- Alternative 4: Dredging the marina to an elevation of -10 ft. MLLW.
- Alternative 5: Combination of Alternative 1 and Alternative 4.
- Alternative 6: Combination of Alternative 3 and Alternative 4.

The results indicate that the long straight breakwater (Alternative 1), and the angled breakwater (Alternative 3), are the most effective at reducing the presence of short-period breaking waves in the marina. Straight breakwater alternatives (Alternatives 1 and 2) do not reduce the long-period water level fluctuations which depend mostly on the entrance cross-sectional area. The alternatives with narrower entrances (Alternatives 3 and 3A) significantly reduce the long period water level fluctuations and associated currents inside the harbor, as well as the short-period breaking waves, but may complicate navigation in the entrance.

Dredging the marina (Alternative 4) reduces the short-period breaking waves in the western most/first leg (worst location), but results in an increase in long-period and shorter-period wave heights in the inner areas of the marina. Dredging combined with a long straight breakwater (Alternative 5 and 6) is effective in some areas for both long and short waves, but causes an increase in wave heights in the inner areas of the marina.

Conceptual-level cost estimates were developed for all 7 alternatives. The costs varied from $162,000 for Alternative 3A (breakwater extension in shallowest depth) to $880,000 for Alternative 5 (Longest breakwater extension combined with marina dredging).

Alternatives were qualitatively compared first evaluating them according to three primary criteria, including 1) reduction in long period surge wave heights, 2) reduction in short period breaking wave heights, and 3) capital construction costs. Next the alternatives were rated for each of the three criteria as excellent, good, moderate, or poor. The alternatives analysis indicates that Alternative 3 is the best performing alternative based on the criteria considered in the analysis. Other criteria and considerations may exist that should be considered during design.

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January 29, 2015

1 INTRODUCTION

This report describes the analysis conducted by Coast & Harbor Engineering, (CHE), a division of Hatch Mott MacDonald, for the Port of Corpus Christi Authority (PCCA) under the Scope of Work in accordance with Master Agreement No. 14-03, Service Order No. 2.

Cline’s Point Marina, located in Port Aransas, has been experiencing detrimental conditions near the marina’s entrance during the passage of deep-draft vessel traffic. The wave activity is generated by large, laden vessels moving outbound in the Corpus Christi Ship Channel (CCSC). The basin was originally protected by an approximately 180 ft. long breakwater constructed in 1976 on the west side of the entrance which by 1980 had deteriorated and partially failed to a 120 ft. long breakwater and continued deteriorating over time, reaching its approximate 60 ft. long current condition, by 2003. The historical aerial photographs of the project site are shown in Appendix D. The deterioration of the west breakwater has reportedly resulted in larger penetration of ship-induced wave activity into the marina. Figure 1 shows the location of the project site and the project vicinity. The marina is located east of Hwy 361 where the Port Aransas Navigation Channel bends westward into Corpus Christi Bay. Figure 2 shows the marina and the existing 60 ft. long west breakwater.

CHE evaluated the mechanisms by which the deep-draft ship pressure field waves enter the harbor, quantified the level of protection afforded by past and present entrance breakwater configurations, and developed conceptual alternatives for improving conditions. This report presents the data collected for the study, ship hydrodynamic modeling results used to evaluate the wave penetration mechanism inside the marina, development and analysis of conceptual alternatives as well as conceptual-level cost estimates.

Figure 1. Project Vicinity.

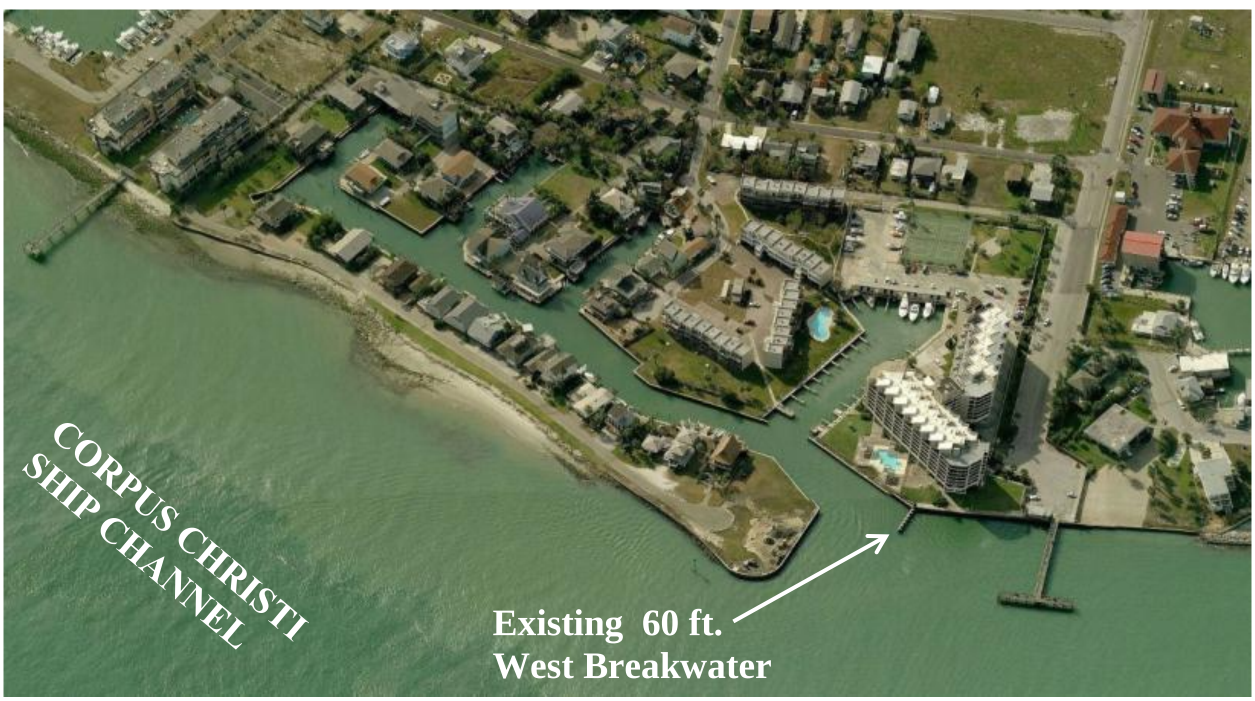

Figure 2. Project Site

2 DATA COLLECTED

Bathymetry data (existing and new) was collected from various sources to develop a numerical modeling grid for evaluation of wave penetration mechanisms and alternatives analysis. The data sources included the Corpus Christi Digital Elevation Model (DEM) (NGDC, 2014), a bathymetry grid developed for a previous project for Marine Sciences Institute (CHE, 2013), and a single-beam hydrographic survey conducted for the analysis in 2014 by Naismith Marine Services (NMS). The Port Aransas Navigation Channel geometry data were obtained from the PCCA and were used to develop an approximate passing vessel route in the numerical model.

The tidal datums used for this project were obtained from the Port Aransas Station published by Texas Coastal Ocean Observation Network (TCOON, 2014) and are shown in Table 1.

Table 1. Tidal datum at Port Aransas station references to NAVD88 datum in feet.

DATUM	ELEVATION [FT. NAVD88]
Mean Higher High Water (MHHW)	1.00
Mean High Water (MHW)	0.96
Mean Sea Level (MSL)	0.62
Mean Low Water (MLW)	0.09
Mean Lower Low Water (MLLW)	-0.04

3 SHIP HYDRODYNAMIC MODELING

Deep-draft vessel-induced surge analyses were performed to provide an evaluation of water level fluctuations and surge-induced current velocities generated by passing vessels inside the marina. The VH-LU model was used to simulate water level and velocity fluctuations generated in the marina by passing deep-draft ships.

3.1 Grid Setup

The bathymetry grid for the VH-LU model was created by merging all pertinent bathymetric data with priority given to the most recent bathymetry data set (NMS). A grid of approximate size 1.2 miles by 0.7 miles with uniform grid spacing of 6.6 ft (2m) was created for the numerical modeling. The bathymetry contours within the modeling grid are shown in Figure 3.

3.2 Input Parameters

The design vessel used for the numerical modeling was the Godavari Spirit, a Suezmax class crude oil tanker with length, beam, and draft of 900ft, 157ft and 45ft, respectively. The design passing vessel was coordinated in advance with PCCA and is representative of the largest tanker that travels within the Corpus Christi Ship Channel. The route used for the vessel movement was outbound (traveling out of Port Aransas to the Gulf of Mexico). Three vessel speeds (6, 8 and 10 knots) were chosen for initial testing. These speeds are representative of normal vessel operation within this reach of the CCSC.

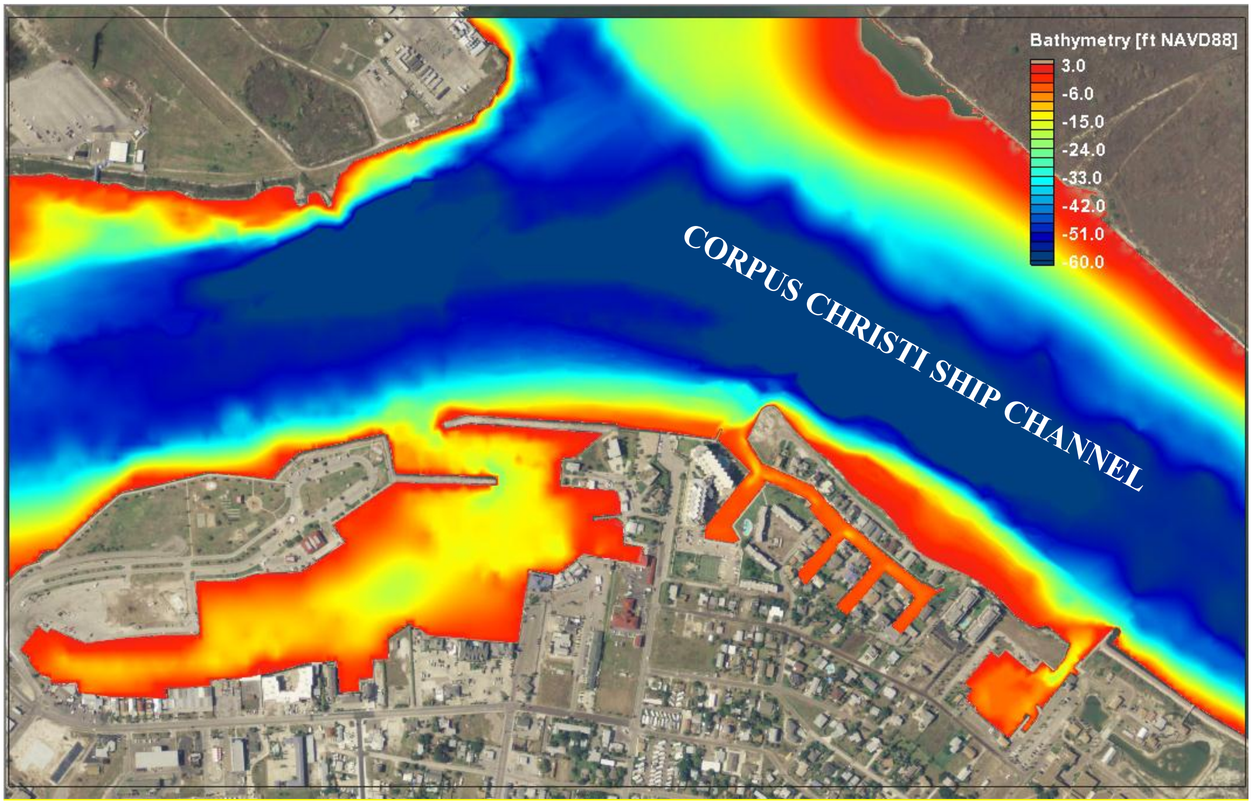

Figure 3. Bathymetry contours for the VH-LU modeling grid for existing conditions.

3.3 Results

Results of the model include water level fluctuations and depth-averaged current velocities. Water surface elevation time series were extracted at three locations (Points A, B & C shown in Figure 4) inside the first/westernmost leg of the marina for the purpose of cursory model qualitative validation. The extracted water level time series were compared with visual observations to get a qualitative sense whether the model was able to reproduce field conditions. The water level time series along with field observation photographs are depicted in Figure 4 through Figure 6. The results indicate an increase in the magnitude of surge wave heights as we move towards the interior of the first leg of the marina from Point A to Point C, due to the increase in wave shoaling, breaking and reflection. This result is also depicted in the field photographs.

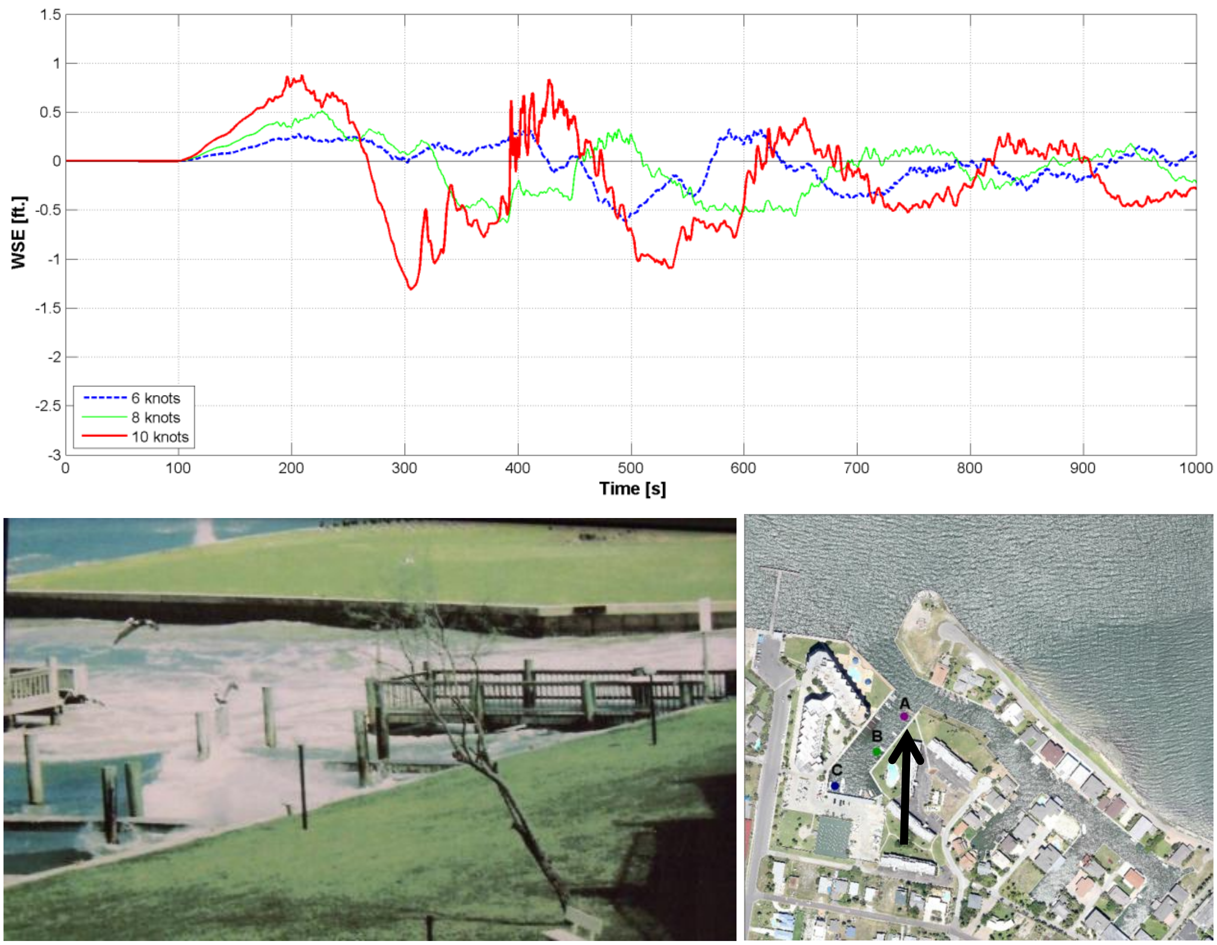

Figure 4. Modeled water level time series at Point A (top) and field observation of breaking wave activity in the same area (bottom-left).

Another modeling result is that the pressure field effects change significantly at higher passing speeds (e.g. 10 knots). At higher speeds, along with the long-period (150-250 sec) surge wave component, additional short-period (high-frequency, 10-20 sec) wave components are observed in the water level time series, that manifest themselves as breaking waves as observed in the site photos.

While it would be preferable to reduce both the long-period and short-period wave components to improve conditions inside the marina, the short-period waves have the potential to cause greater damage to the berthed vessels than the long-period surge waves as long as sufficient underkeel clearance is maintained in the marina. Therefore the reduction of short-period breaking waves is more important than the reduction of long-period surge waves.

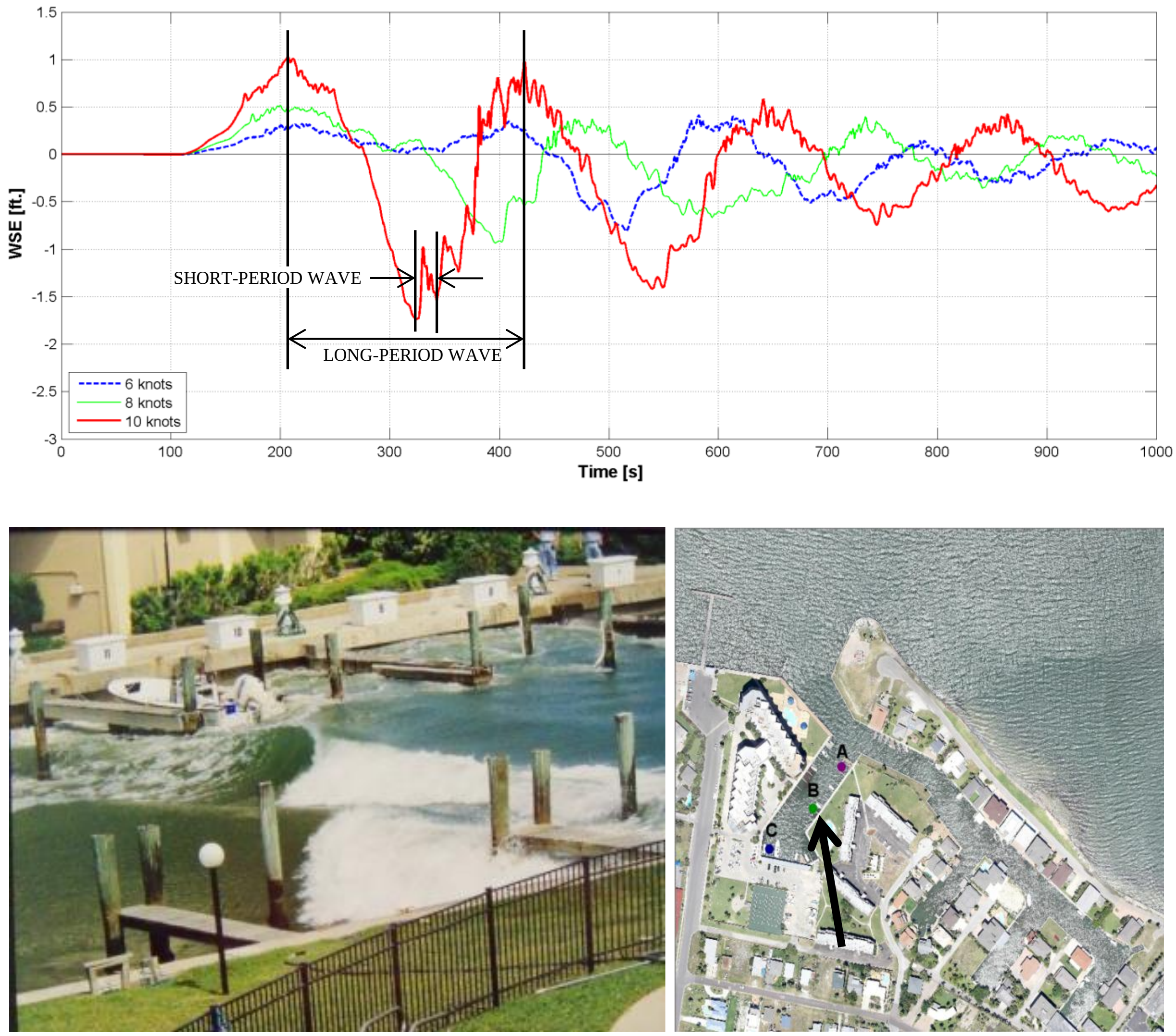

Figure 5. Modeled water level time series at Point B (top), field observation of breaking wave activity in the same area (bottom-left) and location of field observation (bottom-right)

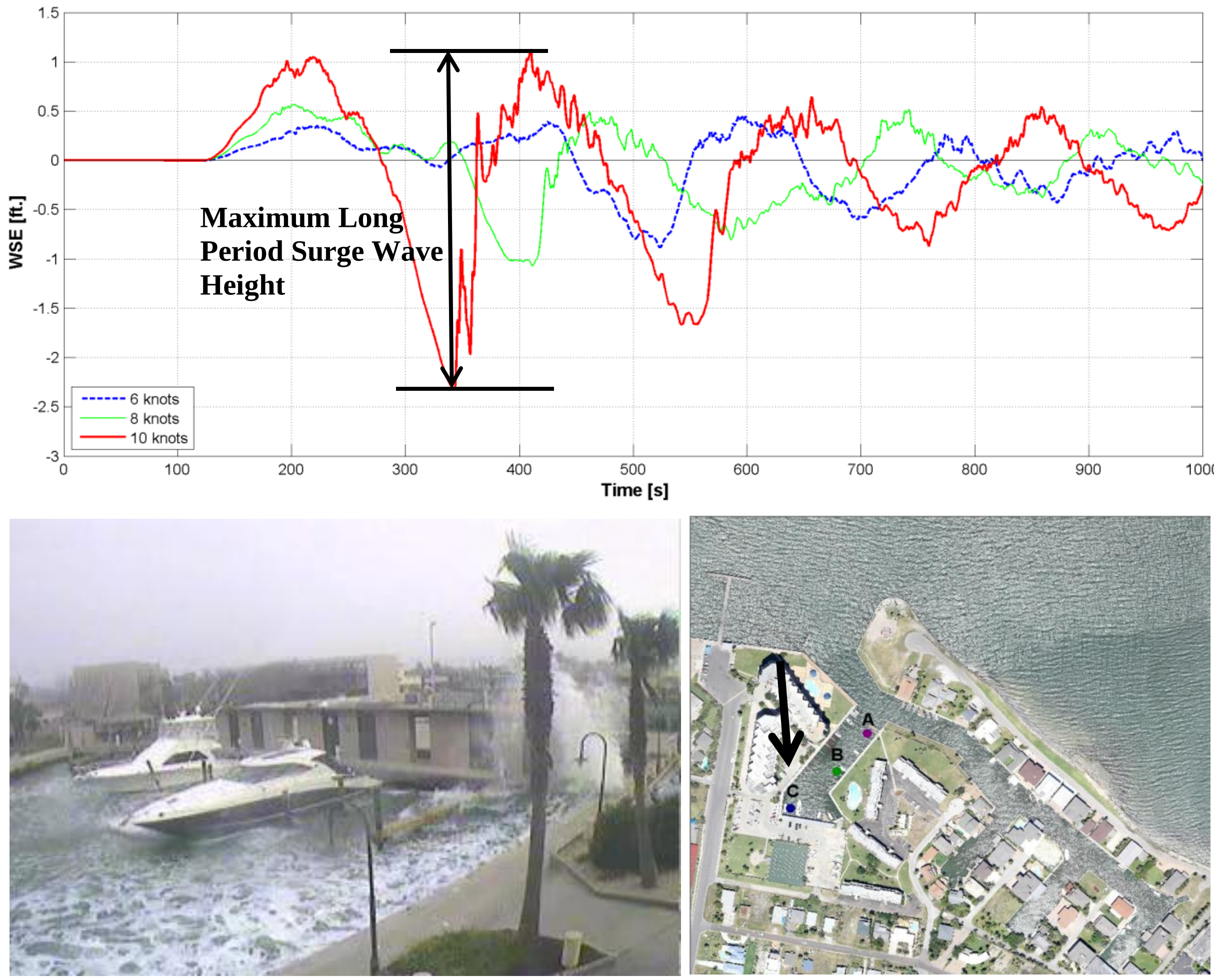

Figure 6. Modeled water level time series at Point C (top), field observation of breaking waves slamming into bulkhead in same area (bottom-left), and location of field observation (bottom-right)

Spatial plots of maximum surge (long period) wave height, significant wave height of the short-period breaking waves, and maximum current velocity were created and are shown in Figure 7. The spatial plot for the short period breaking waves (Figure 7 top-right) was created by isolating the high-frequency component of the waves from the overall water level time series using spectral analysis and then computing the significant wave height for each point with in the modeling grid. It should be noted that the accuracy of the significant wave height calculations is limited due to the short duration of the time histories available, but the relative comparison between wave heights for different alternatives is considered reliable.

The spatial plot for the long period surge waves (Figure 7 top-left) shows the wave heights (water level oscillations) are highest in the first (westernmost) and the last (easternmost) leg of the marina with increasing wave heights as we move towards the interior of the first leg of the marina. For the 10 knots passing vessel scenario, wave heights as high as 3.4 ft. (at Point C) are computed.

The spatial plot for the short period breaking waves (Figure 7 top-right) shows that significant wave heights decrease moving towards the interior of the marina. For the 10 knots passing vessel scenario, breaking wave heights are high near Point C which is indicated in the site photos in Figure 6.

Since the maximum velocity is a manifestation of both the breaking waves and long-period surge waves, the spatial velocity plots (bottom of Figure 7) resemble the overall pattern observed for wave heights.

It can be concluded from these results that the model is able to replicate the observed conditions inside the marina. At higher passing vessel speeds, the pressure field changes significantly as high frequency wave components are also observed. Since the high frequency breaking wave components may be the more problematic component, modeling runs for the alternatives analysis were performed using the 10 knot passing speed.

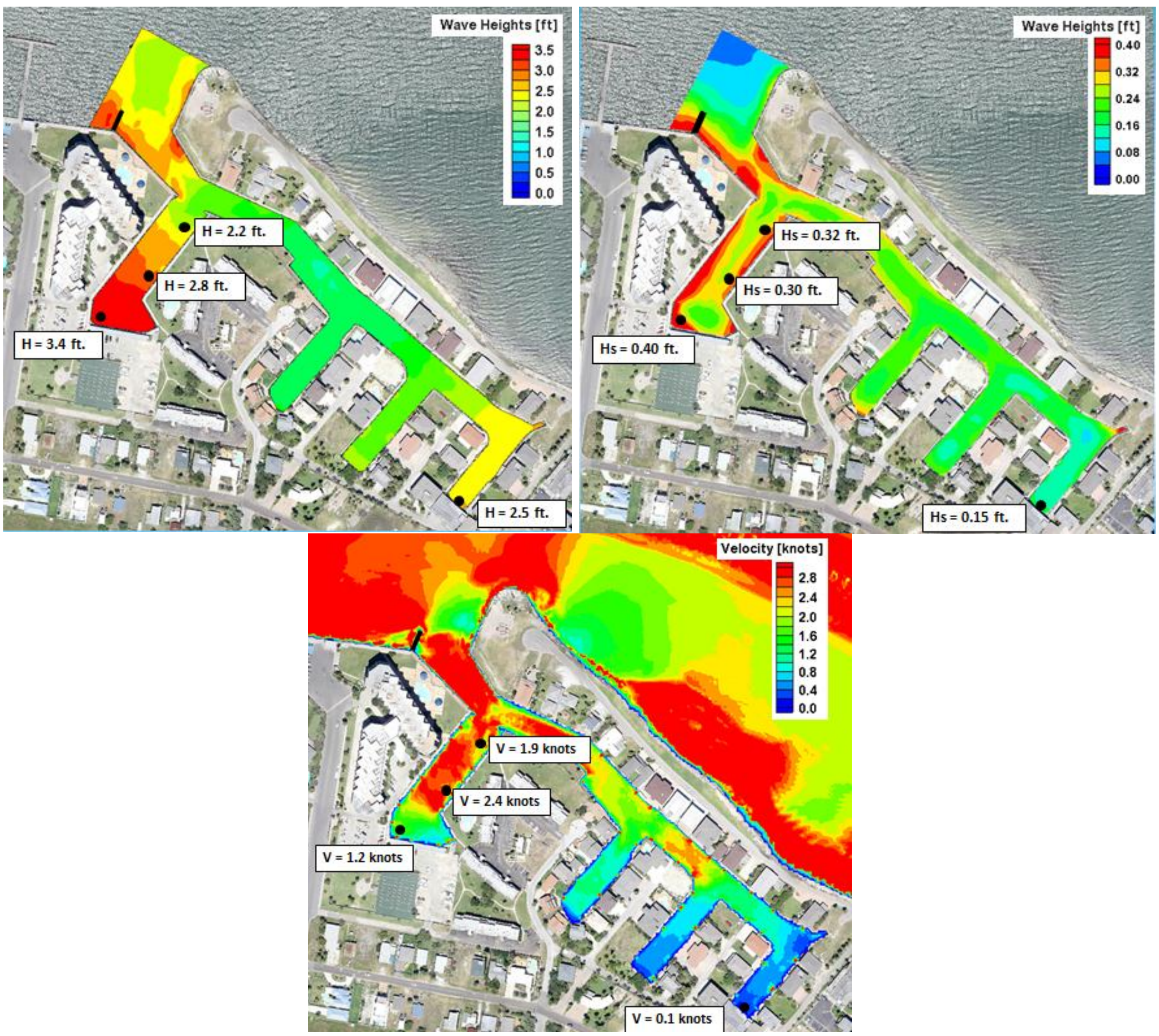

Figure 7. Spatial plots for long period surge waves (top-left), short period breaking waves (top-right) and maximum velocity (bottom) for existing conditions (Alternative 0) with passing vessel at a speed of 10 knots.

4 ALTERNATIVES DEVELOPMENT AND ANALYSIS

Breakwater and dredging alternatives were developed in order to reduce penetration/formation of short-period breaking waves as well as the long-period surge waves. The alternatives consisted of various entrance breakwaters, interior dredging and combinations of both. The following alternatives were developed in coordination with the PCCA:

- Alternative 0: Existing condition, the 60 ft. breakwater
- Alternative 1: Add 120 ft. of new breakwater to and at same orientation as existing breakwater.
- Alternative 2: Add 60 ft. of new breakwater to and at same orientation as existing breakwater).
- Alternative 3: Add 74 ft. of new breakwater to and angled from the existing breakwater.
- Alternative 3A: 66 ft. of new breakwater angled from the eastern bulkhead opposite the existing breakwater.
- Alternative 4: Dredging the marina to an elevation of -10 ft. MLLW.
- Alternative 5: Combination of Alternative 1 and Alternative 4.
- Alternative 6: Combination of Alternative 3 and Alternative 4.

Existing conditions is denoted as Alternative 0. Alternative 1, shown on the left of Figure 8, adds an additional 120 ft. of breakwater in addition to the existing 60 ft. of breakwater, thereby extending the overall breakwater length to 1976 breakwater conditions. The average bottom elevation along the breakwater is approximately -18 ft. NAVD88. Alternative 2, shown on the right of Figure 8, adds 60 ft. of new breakwater (half the length added in Alternative 1) along an approximate bottom elevation of -14 ft. NAVD88.

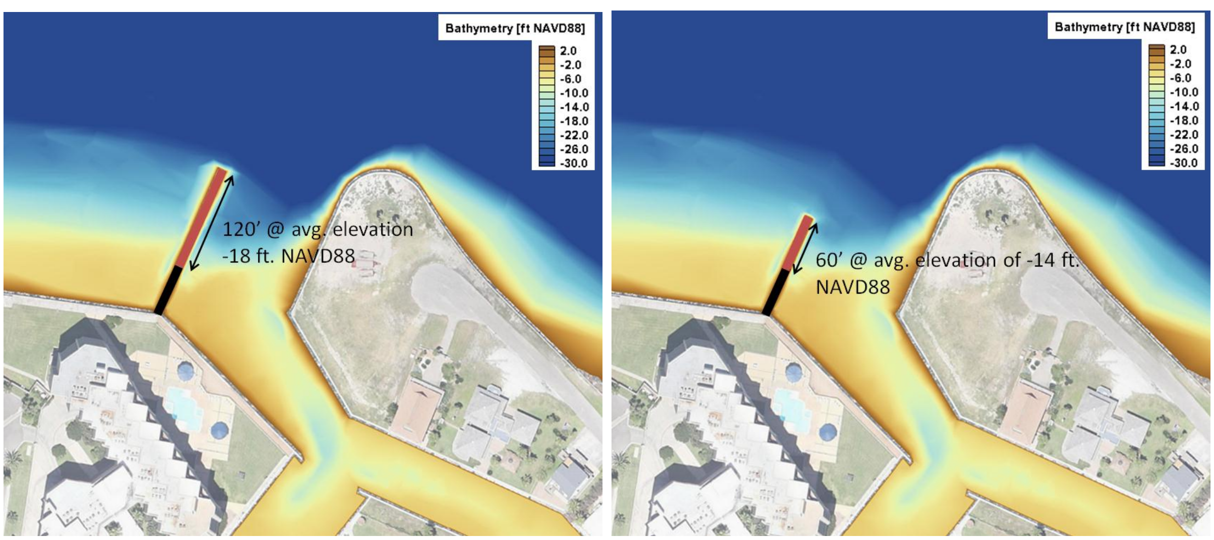

Figure 8. Conceptual representation of Alternatives 1 (left) and 2 (right).

Alternative 3, shown on the left of Figure 9, adds approximately 74 ft. of new breakwater angled eastward from the tip of the existing breakwater so that the marina entrance width is reduced to approximately 75 ft. The average bottom elevation along this alignment is approximately -9 ft.

NAVD88. It should be noted that the marina opening for this alternative is aligned with the deepest entrance depths that are present under existing conditions. Alternative 3A, shown on the right of Figure 9, adds an angled breakwater with average bottom elevation of -6 ft. NAVD88 starting from the eastern bulkhead. The marina opening is still 75 ft. wide but the entrance is west of the existing deepest entrance depths to the marina and therefore some additional initial entrance dredging may be required.

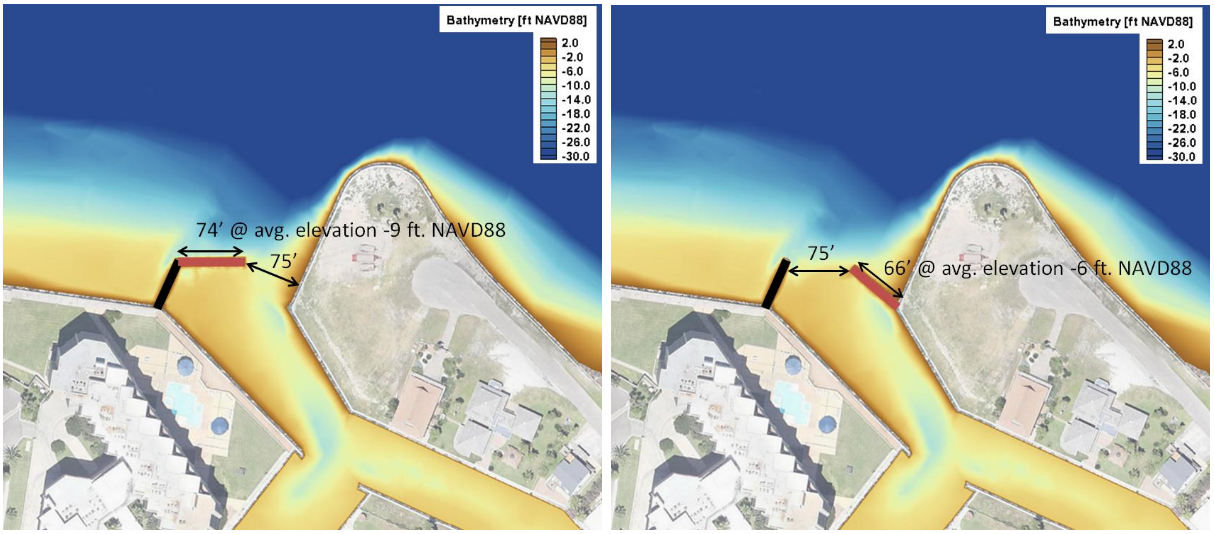

Figure 9. Conceptual representation of Alternatives 3 and 3A.

Alternative 4, shown in Figure 10, did not incorporate any modification to the existing breakwater but included dredging the fairways of the marina to an elevation of -10 ft. MLLW from the existing average elevation of approximately -7 ft. MLLW. Alternatives 5 and 6 (shown on the left and right of Figure 11, respectively) are alternatives that combined dredging (Alternative 4) with the entrance modification alternatives. Alternative 5 consisted of a combination of Alternative 1 and Alternative 4, and Alternative 6 consisted of a combination of Alternative 3 and Alternative 4.

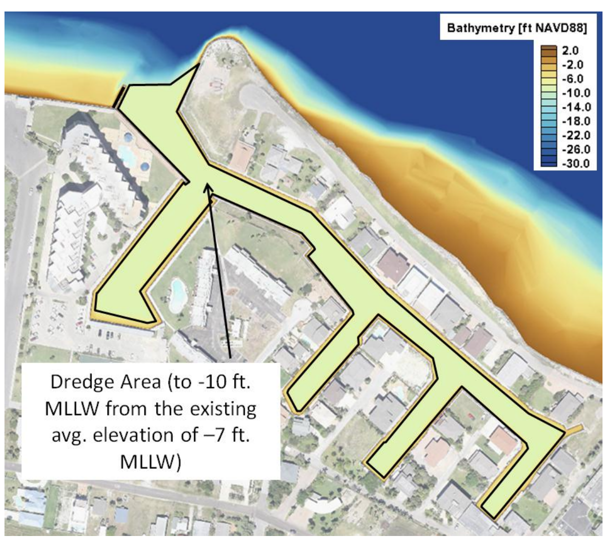

Figure 10. Conceptual representation of Alternative 4.

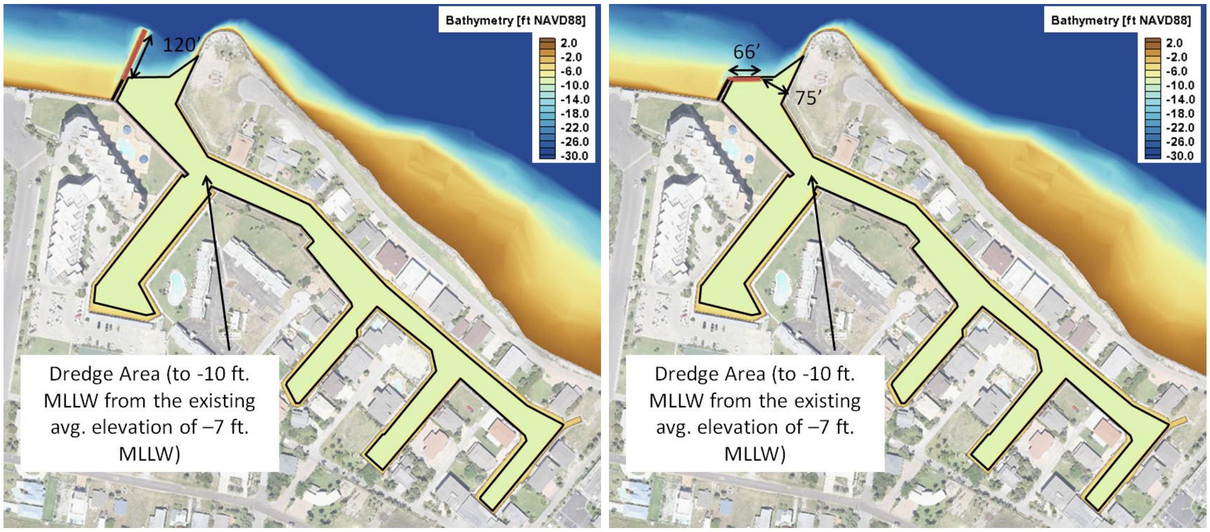

Figure 11. Conceptual representation of Alternatives 5 and 6.

All alternatives were simulated using the same tanker (Godavari Spirit), traveling the same outbound route as was used for existing conditions, with a speed of 10 knots. Maximum surge wave height, significant wave height for the short period breaking waves and maximum current velocity were calculated for all of the alternatives in a manner identical to calculations performed for existing conditions (Figure 7), and the results are shown in Appendices A-C. In addition to the spatial plots for each alternative, spatially variable differences in maximum surge wave height and significant wave height for the short period breaking waves over existing conditions were calculated and are plotted in Appendices A and B, respectively.

The results show that the long, straight breakwater (Alternative 1) and the angled breakwater (Alternative 3) are most effective in reducing the presence of short-period breaking waves in the marina (Figures B1 and B3, respectively). However, the straight breakwater alternatives (Alternative 1 and 2) do not reduce the long-period water level fluctuations (Figures A1 and A2), which depend mostly on the entrance cross-sectional area. The alternatives with narrower entrances (Alternative 3 and 3A) reduce both the long period surge waves and short period breaking waves significantly (Figures A3 and A4 and B3 and B4, respectively).

Maximum current velocity inside the marina for each of the alternatives was calculated and is shown in Figures C1 through C8. These velocities are generated by both the long-period surge waves and short-period breaking waves. It should be noted that the locations where maximum velocities occur inside the marina changes under different alternative configurations. The results show that for existing conditions, the maximum velocity generated by the passing vessel at 10 knots is 4.6 knots, and is similar or less for all alternatives except the angled breakwater alternatives (Alternative 3 and 3A). For Alternative 3 and 3A, the maximum velocity is increased at the marina entrance by 0.9 to 1.3 knots (over the existing conditions' 4.6 knots) due to the narrowing of the marina entrance.

Dredging the marina (Alternative 4) reduces the heights of the short-period breaking waves in the first leg (worst location under existing condition), but results in an increase in long-period and shorter-period wave heights in the inner areas of the marina (Figures A5 and B5), as shown in Figures B5. Dredging combined with a long straight breakwater (Alternative 5) is effective in most areas for both long and short waves, but causes an increase in wave heights in the inner areas of the marina (Figures A6 and B6). Therefore it appears that dredging the entire marina is not an attractive alternative simply for the purpose of reducing ship-induced wave penetration and transformation within the harbor. However, while not evaluated here, some selective dredging in the marina may be feasible that could reduce the harmful effects of the wave energy while avoiding increases in other areas.

5 CONCEPTUAL-LEVEL COST ESTIMATES

Conceptual-level cost estimating was performed for all alternatives. The cost estimates discussed in this section account for only capital costs, i.e. the initial cost of construction, and does not take into account any maintenance costs. These estimates assume typical equipment is used by contractors who commonly perform marine construction activities in the region.

The breakwater alternatives were assumed to be vertical cantilevered steel sheet pile (AZ 26 – 700N) with concrete cap for their entire lengths. The breakwater top elevation was assumed to be +4 ft. NAVD88 and the design water level was assumed to be +1 ft. NAVD88 (MHHW). The sheet piles were assumed to be epoxy coated on both sides for the full height. Other cost estimate assumptions that were used for the sheetpile only alternatives (Alternative 1, 2, 3 and 3A) are shown in Table 2.

It should be noted that the breakwater alternatives, primarily the crest elevation and placement of concrete cap, were designed to approximately match the existing breakwater. The design of the breakwater should be revisited in the future during preliminary and final engineering design with respect to navigability and other factors.

Table 2. Cost estimating assumptions used for sheetpile alternatives.

	Alternative 1	Alternative 2	Alternative 3	Alternative 3A
Avg. Mudline Elevation (ft.)	-18	-14	-9	-6
Embedment Depth. (ft.)	19	15	10	7
Breakwater Length (ft.)	120	60	74	66

For the dredging alternatives, it was assumed that the fairways in the marina would be dredged to an elevation of -10 ft. MLLW and that appropriate upland disposal will be located within 10 miles of the project site. Other dredging assumptions used for cost estimating are shown in Table 3.

Table 3. Cost estimating assumptions for the dredging alternatives.

	Bucket Size [cy]	Average Mudline Elevation [ft. NAVD88]	Barge Size [cy] (70% Full)	Volume to Dredge [cy]	Production Rate [cy/day]	Number of hrs. in a work day [hrs.]	Total Estimated Days [days]
Dredging Assumptions	5	-7	1,050	20,400	2,520	8	8

Using the above assumptions, conceptual-level cost estimates for different alternatives were developed and are shown in Table 4. It should be noted that the cost estimate for replacing the existing 60 ft. of breakwater with a new sheetpile breakwater with concrete cap (similar to breakwater extension alternatives) was also computed and is shown as the cost under existing conditions. It should be noted that Alternative 3A would likely require additional entrance dredging which is not included in the cost estimates shown below.

Table 4. Conceptual-level cost estimates.

	Breakwater	Dredging	Entrance Width [ft.]	Unit Cost	Total Cost [$]
Existing Condition	Existing	--	130	$2,800/LF	$168,000
Alternative 1	1976 Condition	--	130	$4,085/LF	$490,000
Alternative 2	Halfway to 1976 Condition	--	130	$3,720/LF	$223,000
Alternative 3	Angled from Existing Breakwater	--	75	$2,870/LF	$212,000
Alternative 3A	Angled from Eastern Bulkhead	--	75	$2,460/LF	$162,000
Alternative 4	Existing	-10 ft. MLLW	130	$19/CY	$390,000
Alternative 5	1976 Condition	-10 ft. MLLW	130	--	$880,000
Alternative 6	Angled from Existing	-10 ft. MLLW	75	--	$602,000

In addition to the construction cost, preliminary and final engineering design and permitting costs were also estimated. The total design and permitting costs are estimated to be approximately $60,000 and include the additional coastal engineering analysis tasks in support of the design, preparation of the design plans and memorandum, and development and submission of the permitting package. It is assumed that any new data required for design and permitting purpose (bathymetry/topography, geotechnical, coastal boundary survey, etc.) will be provided and is not accounted for in the above cost estimate. The approximate cost estimate for the data collection effort may range between $25,000 and $75,000 depending on the data required and should be determined during the detailed design and permitting phase.

6 ALTERNATIVES EVALUATION MATRIX

Alternatives were analyzed based on the following three criteria: 1) reduction in long period surge wave heights, 2) reduction in short period breaking wave heights, and 3) capital construction costs. Performance of each alternative with regard to each of the three criteria was classified as excellent, good, moderate, or poor. An evaluation matrix summarizing the qualitative rankings assigned to each of the alternatives is presented in Table 5.

Table 5. Alternatives evaluation matrix

Criterion	Alternative 1	Alternative 2	Alternative 3	Alternative 3A	Alternative 4	Alternative 5	Alternative 6
Surge Wave Height Reduction	Moderate	Moderate	Excellent	Excellent	Poor	Poor	Good
Breaking Wave Height Reduction	Good	Moderate	Excellent	Moderate	Moderate	Moderate	Good
Capital Cost	Poor	Good	Good	Excellent	Moderate	Poor	Poor

7 Conclusions

Analysis results indicate that Alternative 3 is the best overall alternative based on the criteria used in the evaluation. The reduction in both long-period surge (water level changes) and short-period breaking wave effects is superior to all other alternatives. The entrance width of 75 feet is similar to other constrictions within the marina. However, it is understood that restricting the marina entrance width may be a concern and navigation safety should be evaluated and refinements made during design.

8 References

Coast & Harbor Engineering (2013). Boat Basin Rehabilitation Project, University of Texas Marine Sciences Institute, Port Aransas, Texas. Technical Memorandum prepared for Port of Corpus Christi Authority (PCCA).

Fenical, S., Kolomiets, P., Kivva, S., and M. Zheleznyak (2006). Numerical Modeling of the Impacts on Berthed Vessels and Shorelines. International Conference on Coastal Engineering.

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Appendix A

Maximum Surge Wave Height Plots

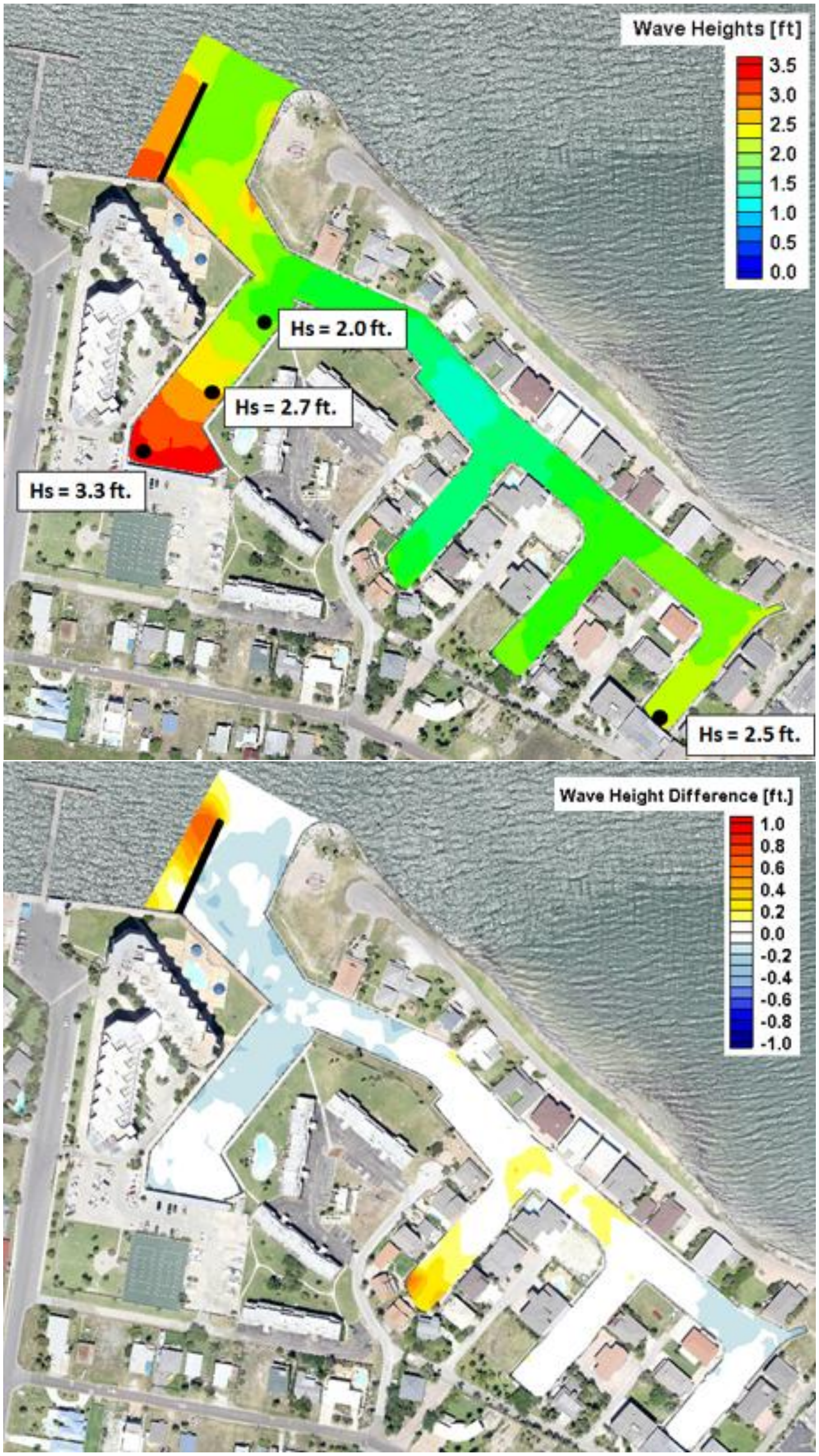

Figure A1. Maximum surge wave height (top) and surge wave height difference from existing (bottom) for Alternative 1.

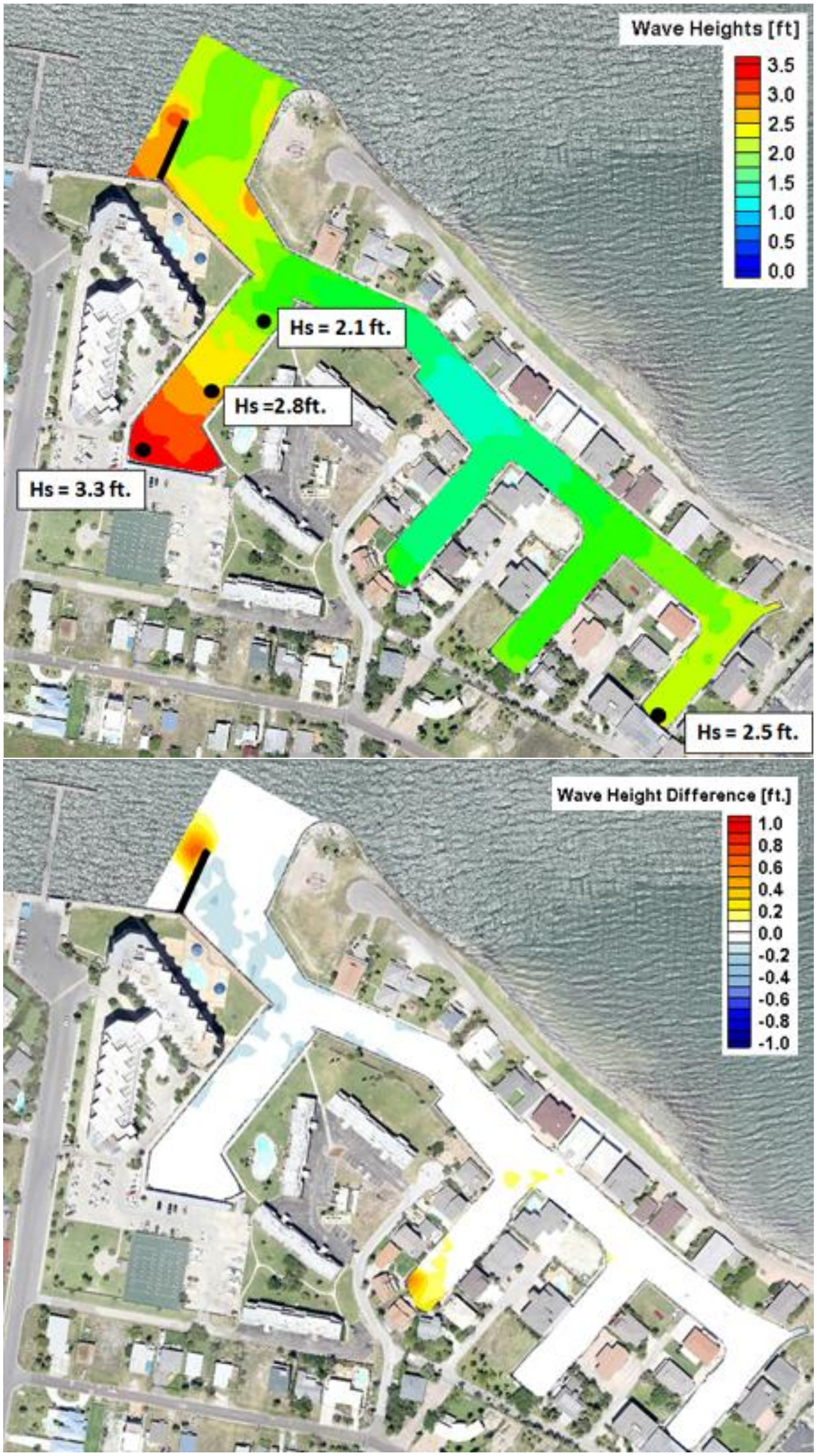

Figure A2. Maximum surge wave height (top) and surge wave height difference from existing (bottom) for Alternative 2.

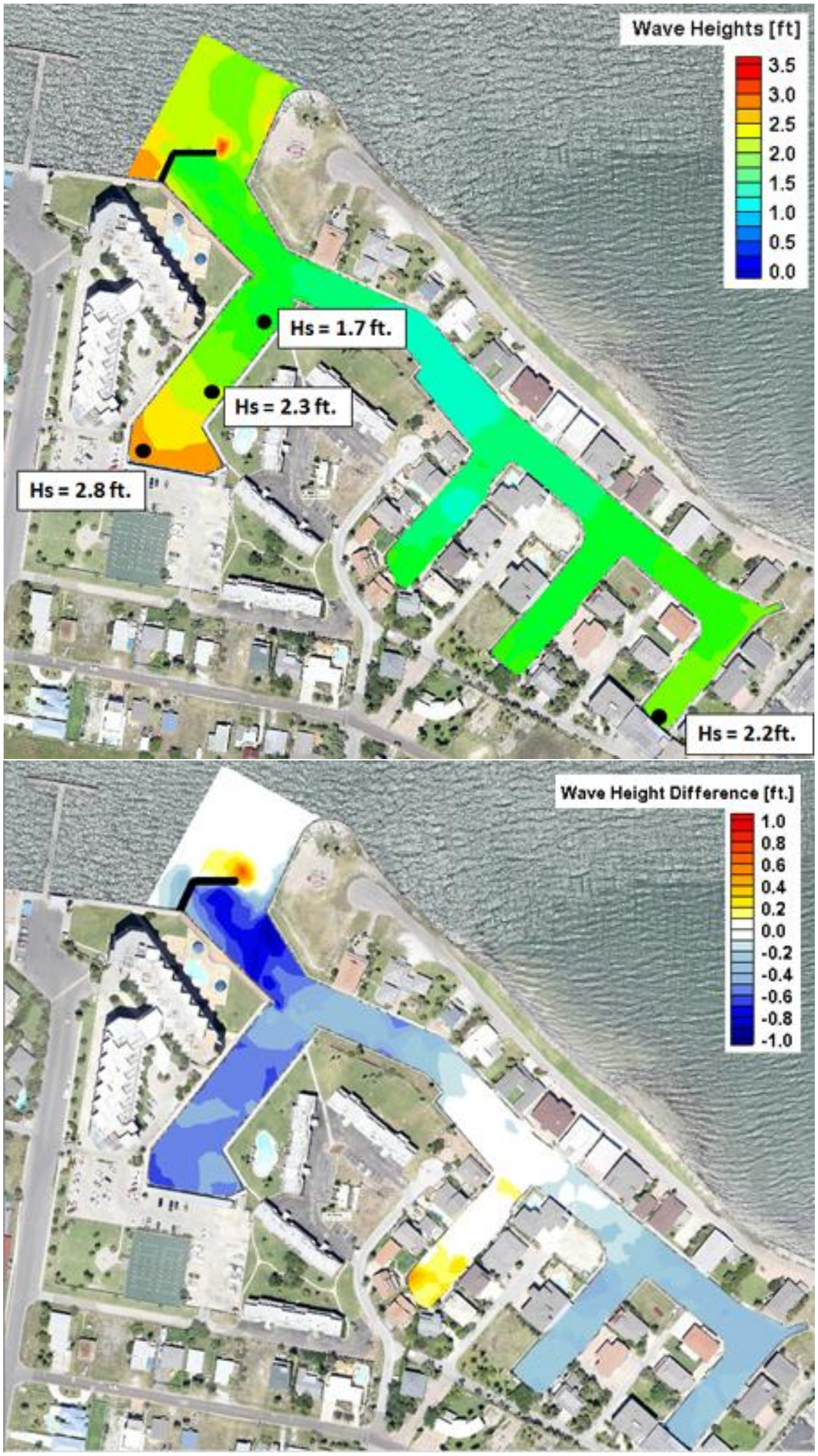

Figure A3. Maximum surge wave height (top) and surge wave height difference from existing (bottom) for Alternative 3.

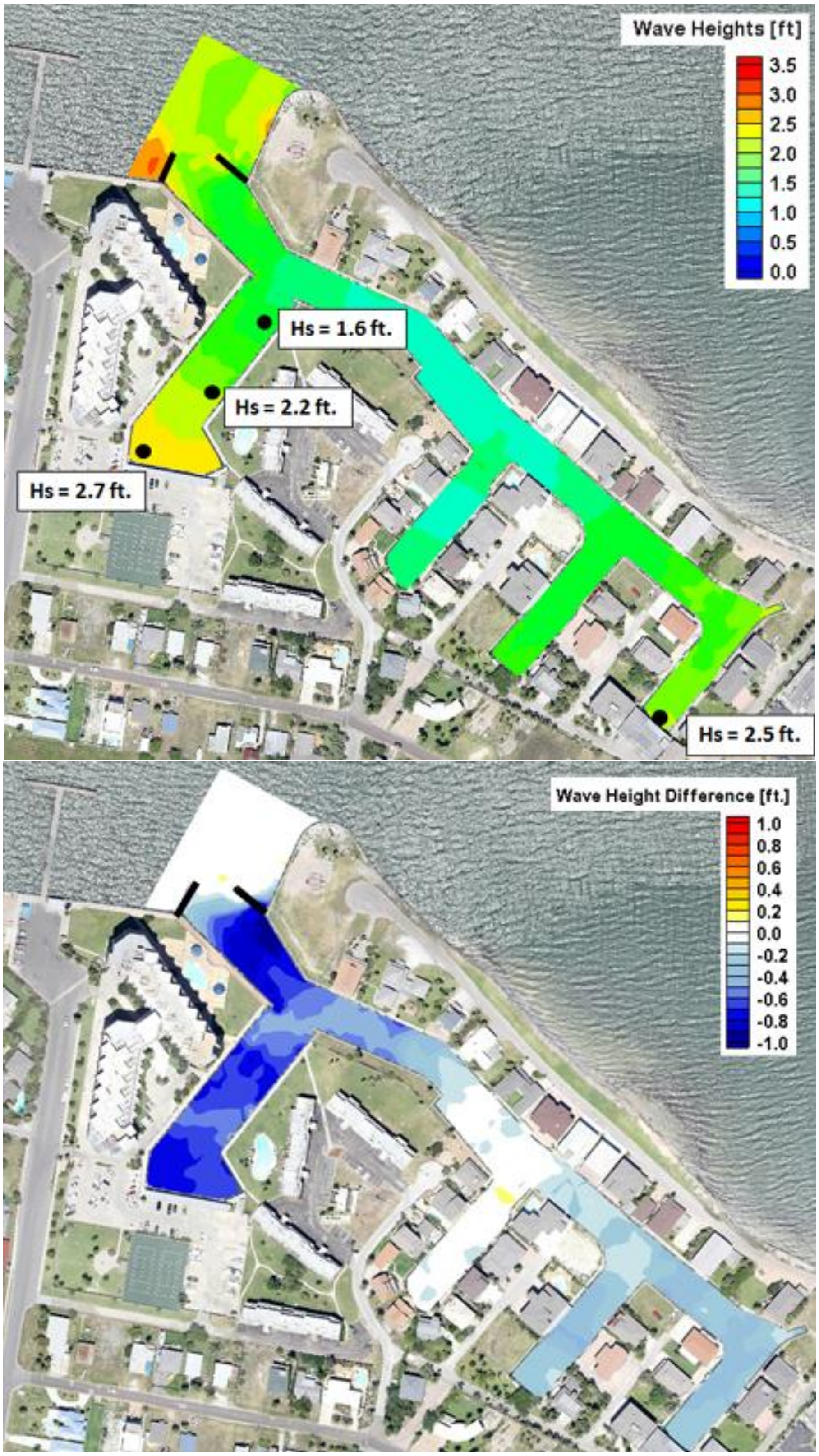

Figure A4. Maximum surge wave height (top) and surge wave height difference from existing (bottom) for Alternative 3A.

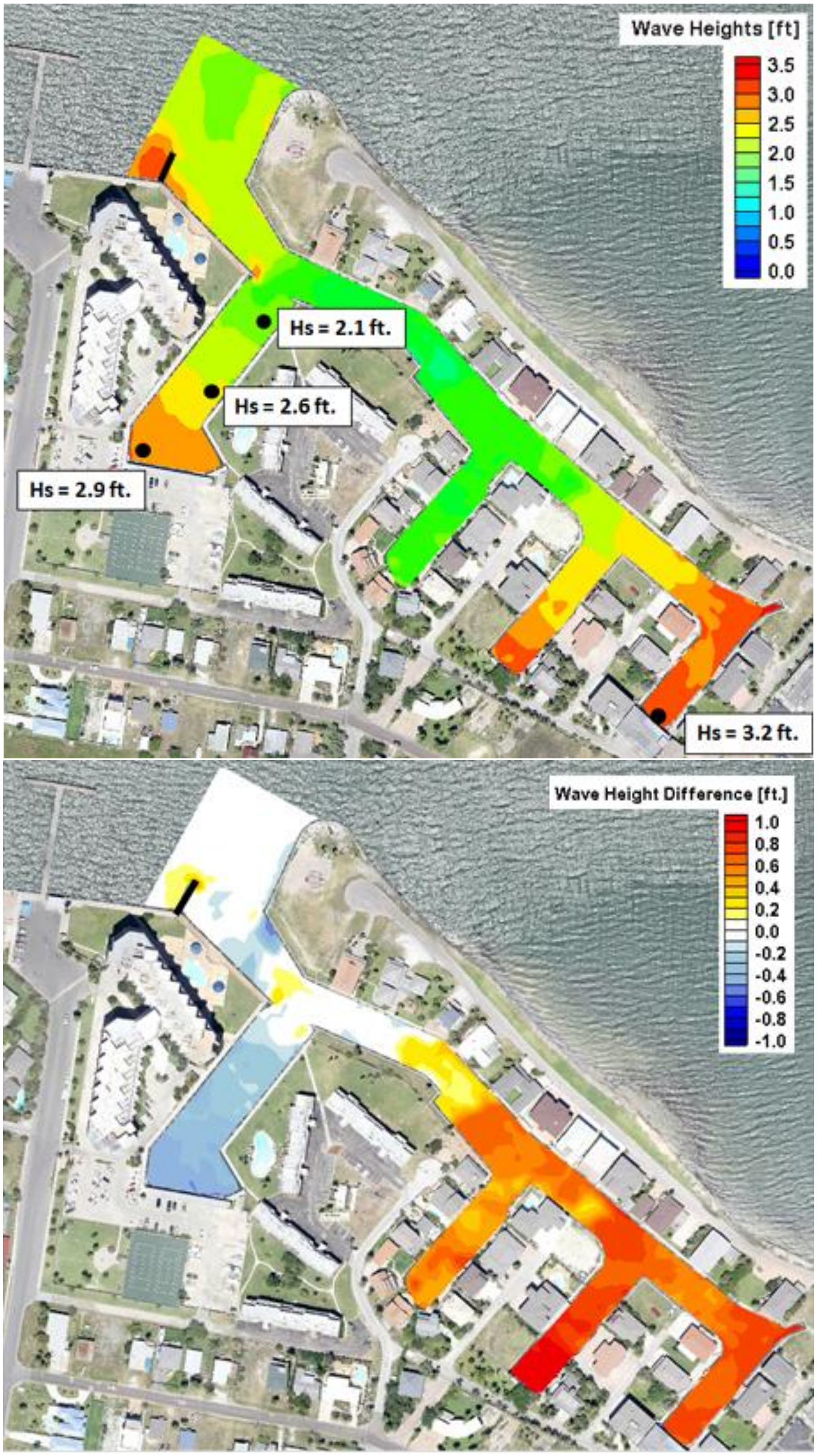

Figure A5. Maximum surge wave height (top) and surge wave height difference from existing (bottom) for Alternative 4.

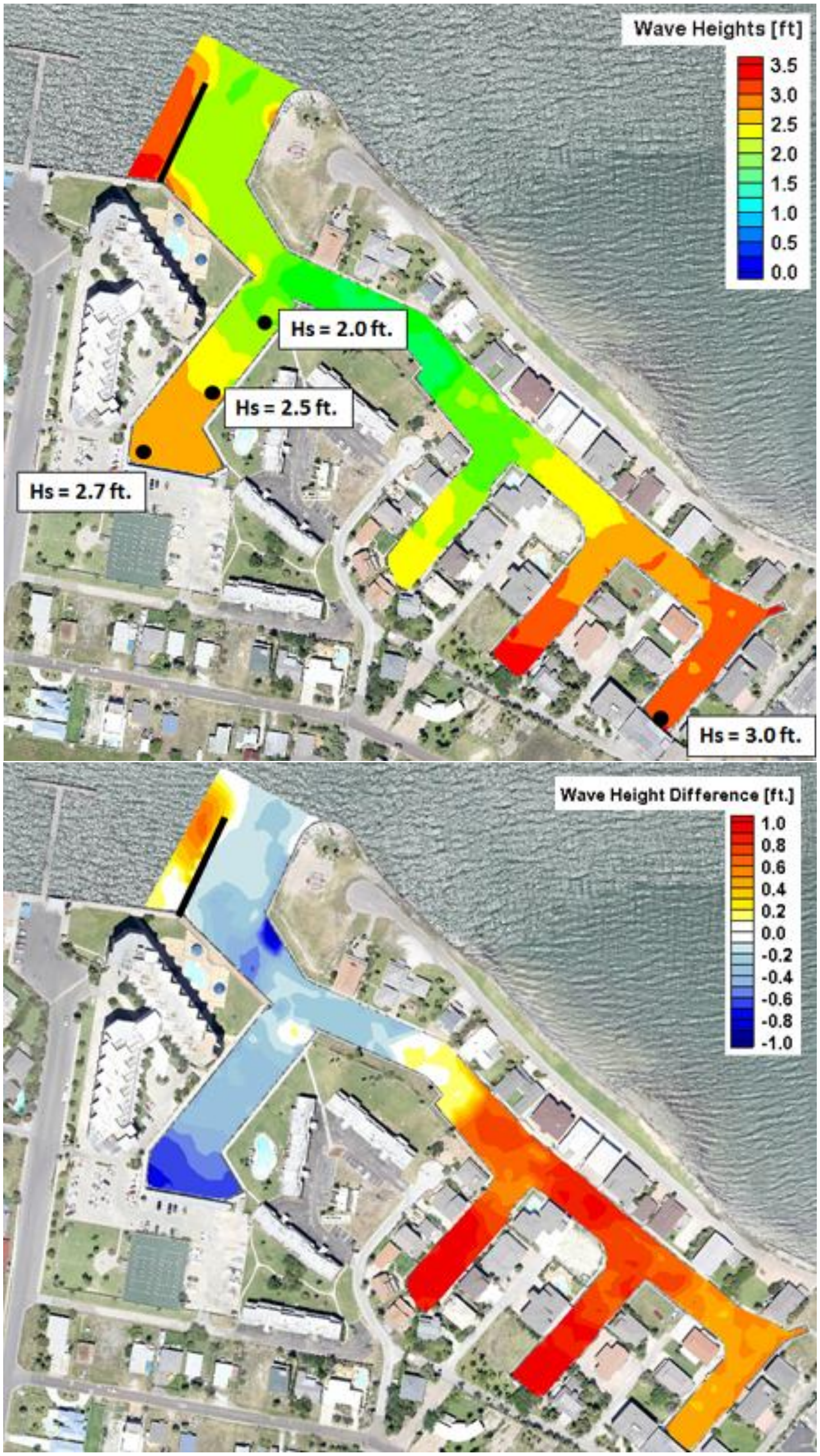

Figure A6. Maximum surge wave height (top) and surge wave height difference from existing (bottom) for Alternative 5.

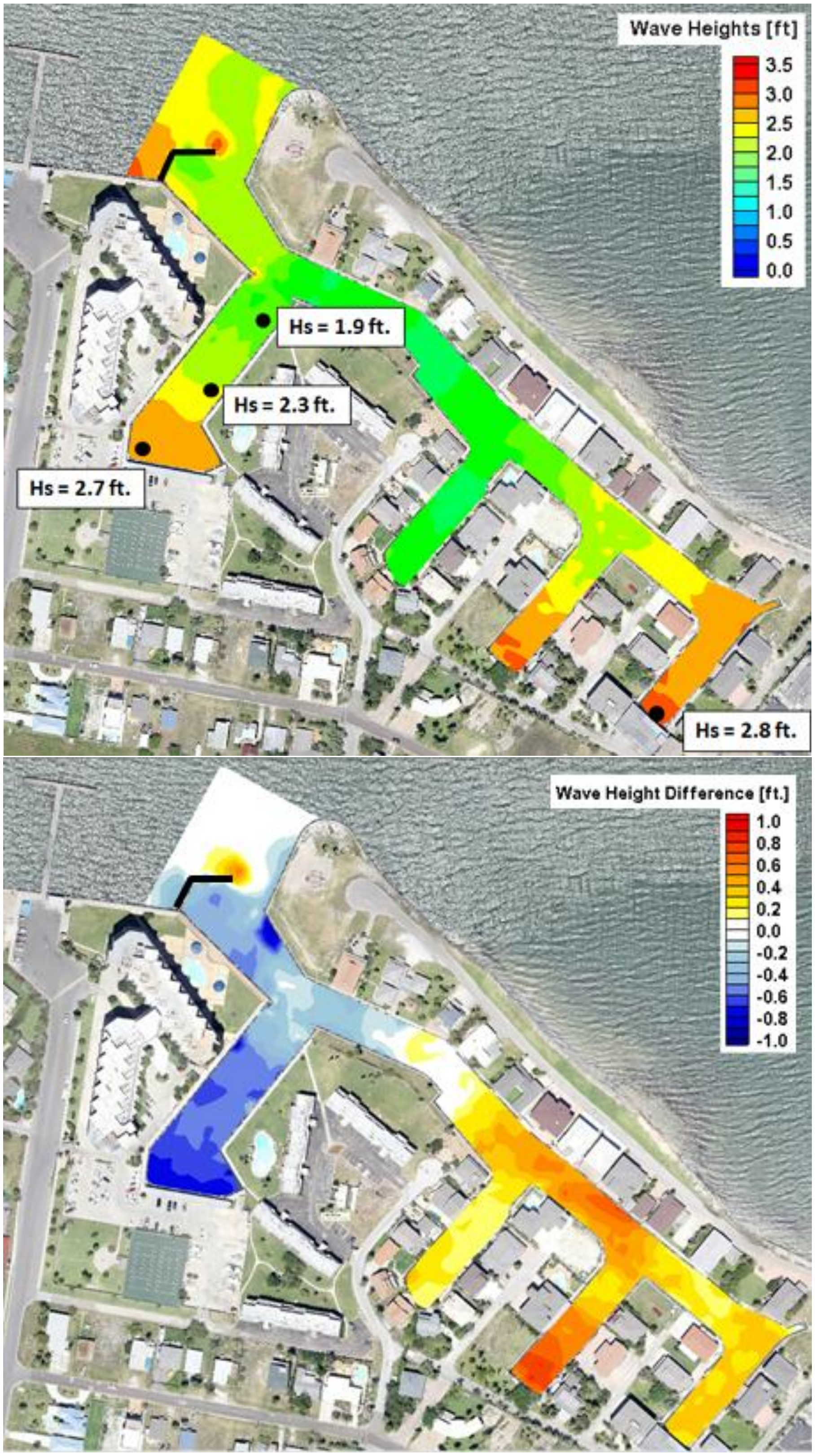

Figure A7. Maximum surge wave height (top) and surge wave height difference from existing (bottom) for Alternative 6.

Appendix B

Breaking Wave Significant Wave Height Plots

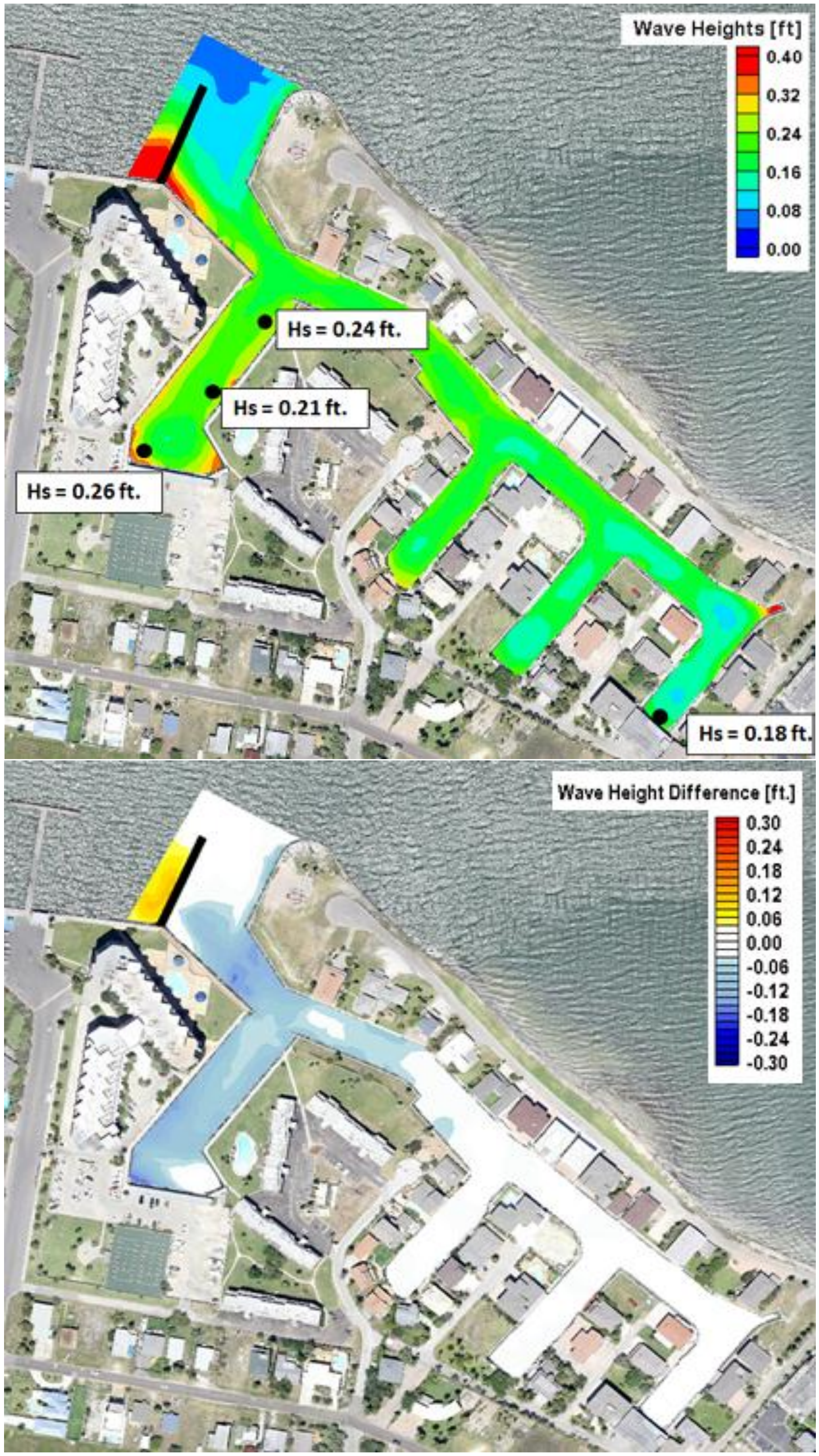

Figure B1. Significant breaking wave height (top) and significant breaking wave height difference from existing (bottom) for Alternative 1.

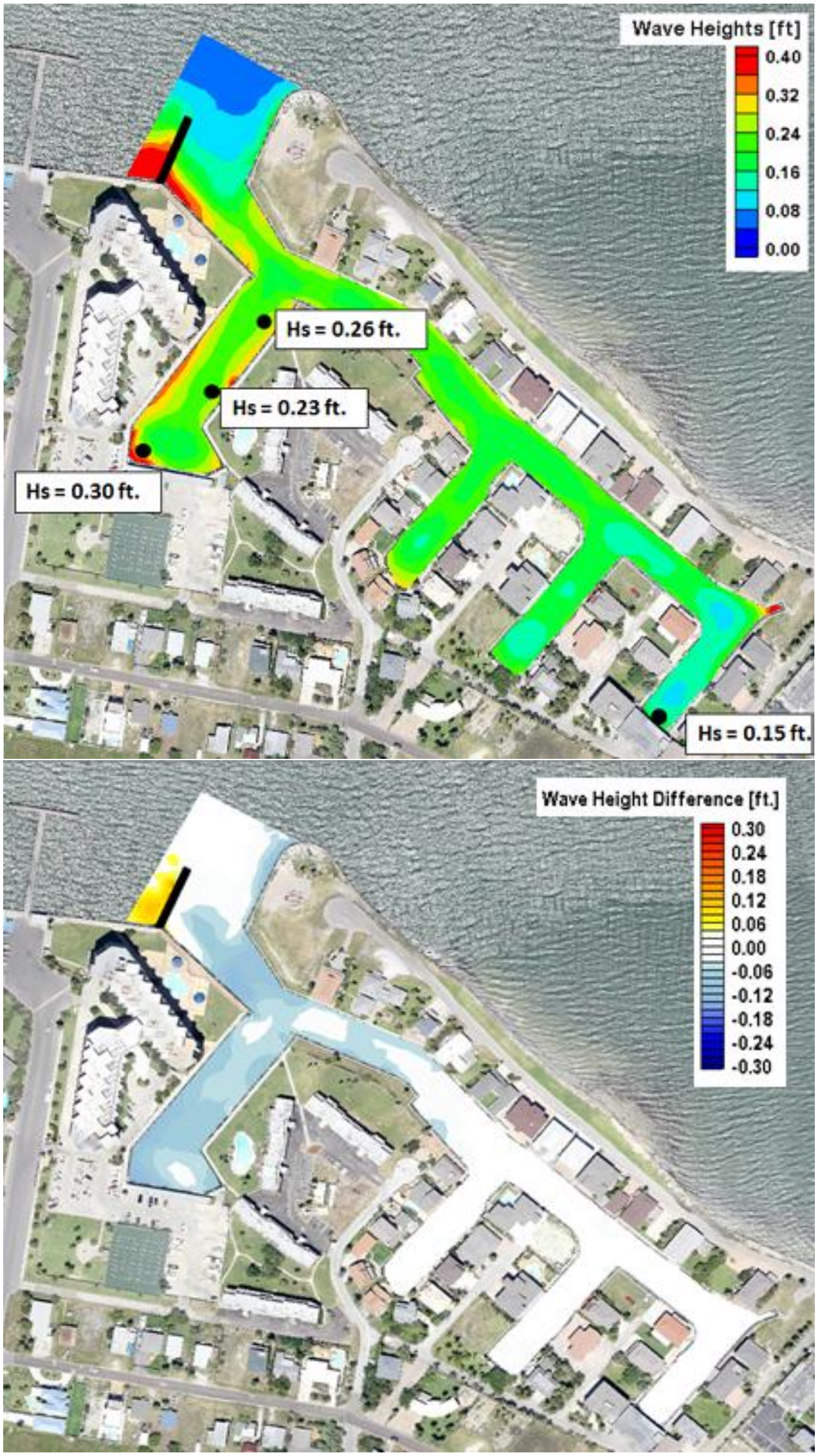

Figure B2. Significant breaking wave height (top) and significant breaking wave height difference from existing (bottom) for Alternative 2.

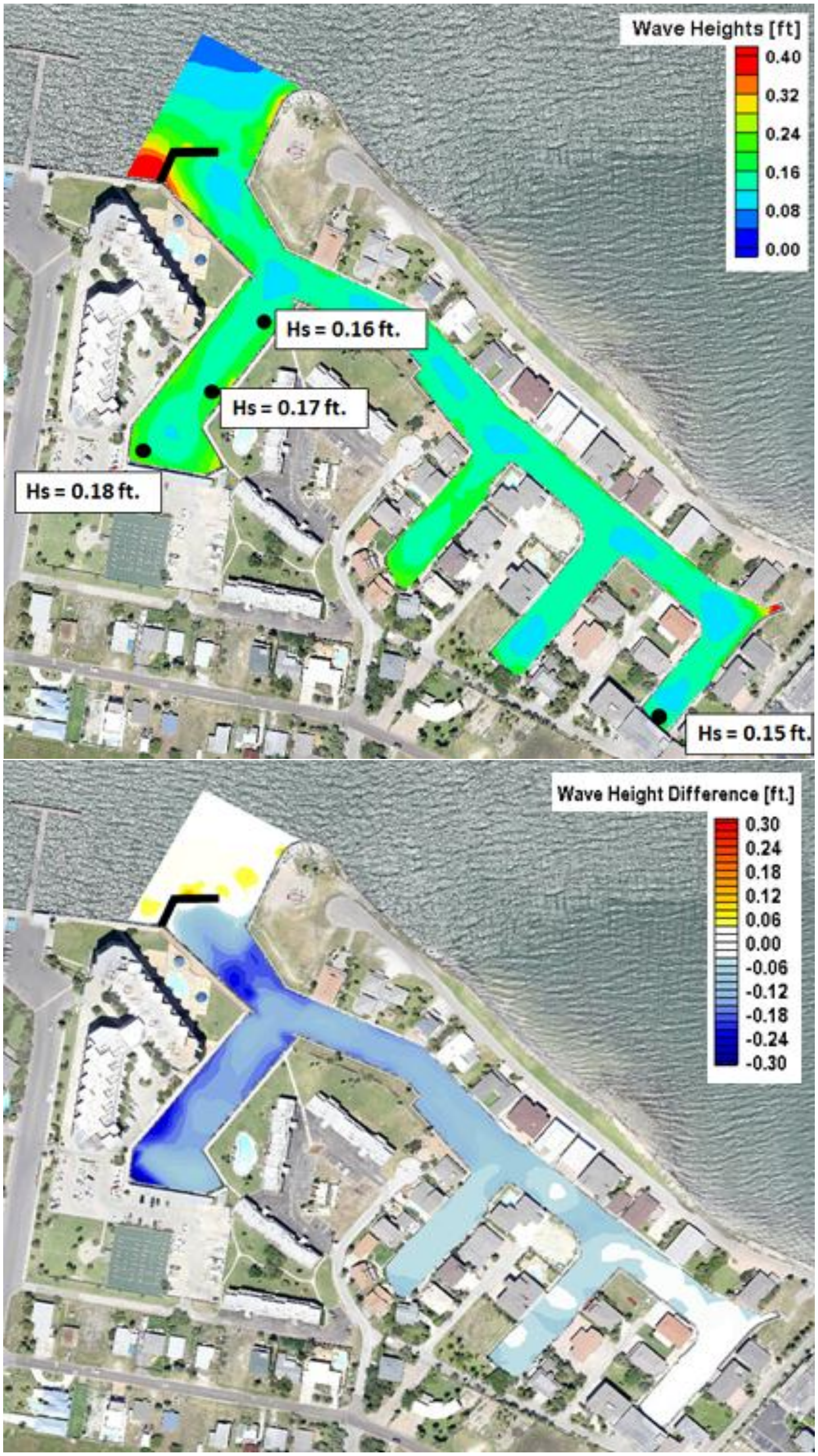

Figure B3. Significant breaking wave height (top) and significant breaking wave height difference from existing (bottom) for Alternative 3.

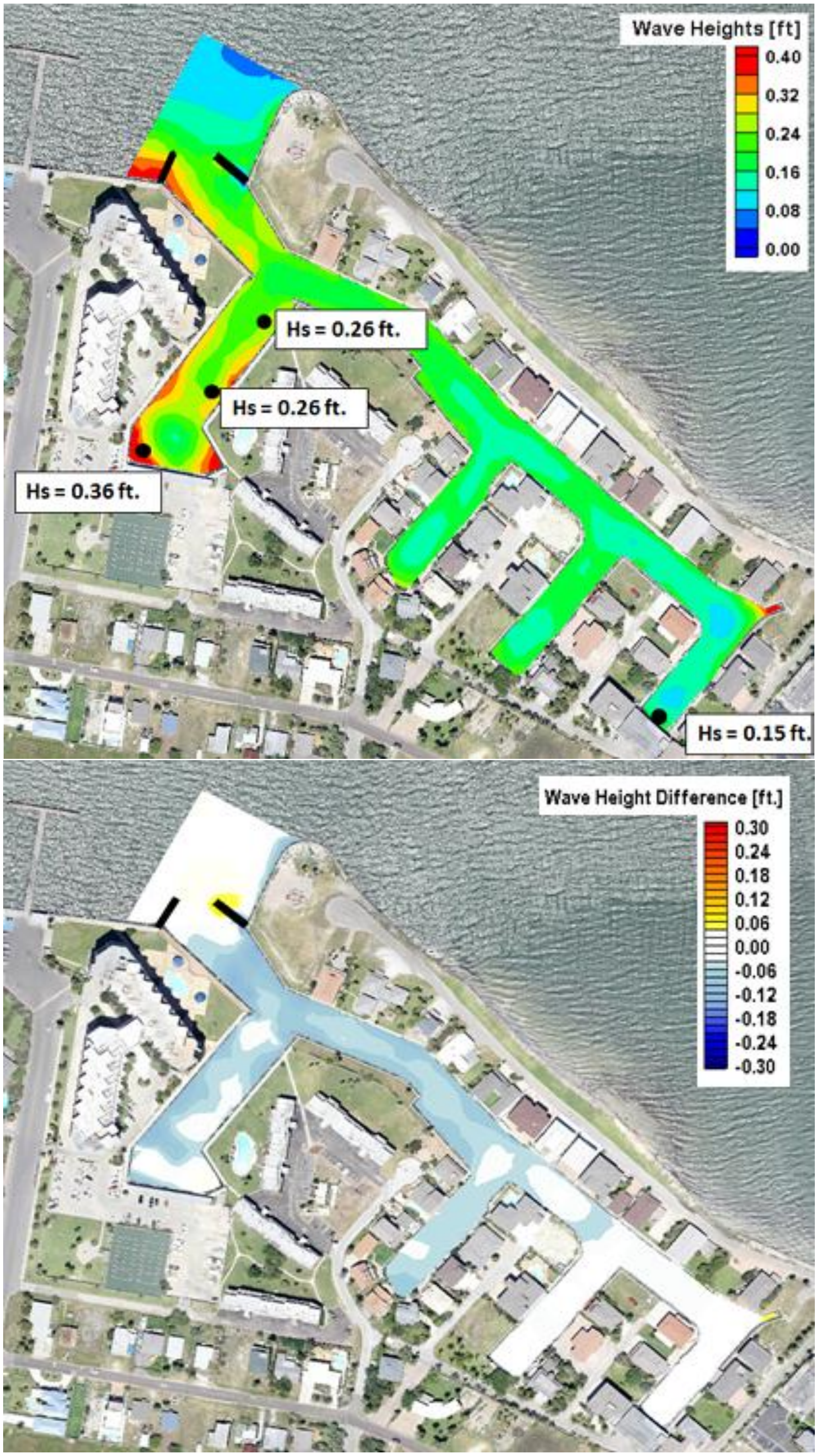

Figure B4. Significant breaking wave height (top) and significant breaking wave height difference from existing (bottom) for Alternative 3A.

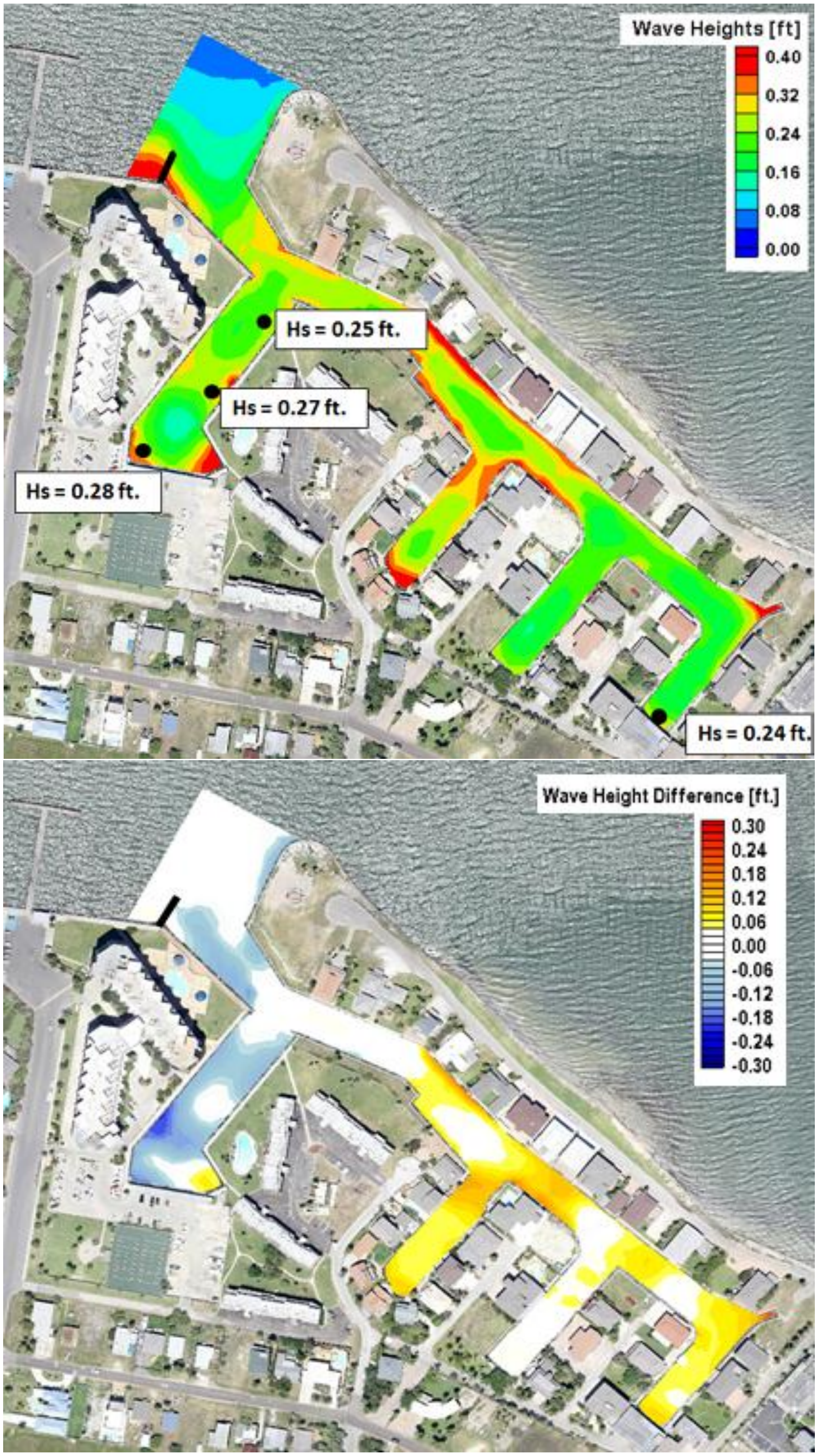

Figure B5. Significant breaking wave height (top) and significant breaking wave height difference from existing (bottom) for Alternative 4.

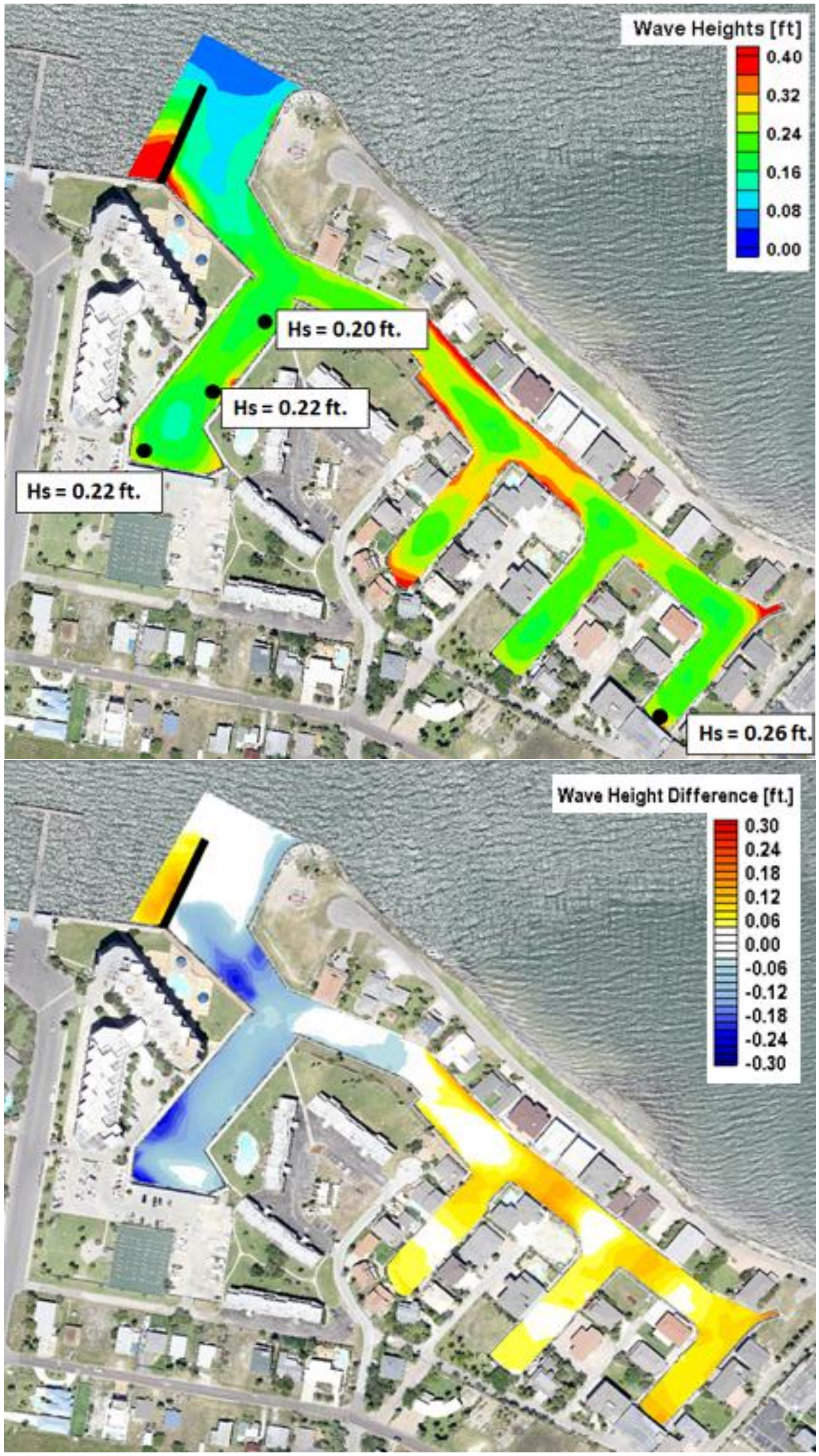

Figure B6. Significant breaking wave height (top) and significant breaking wave height difference from existing (bottom) for Alternative 5.

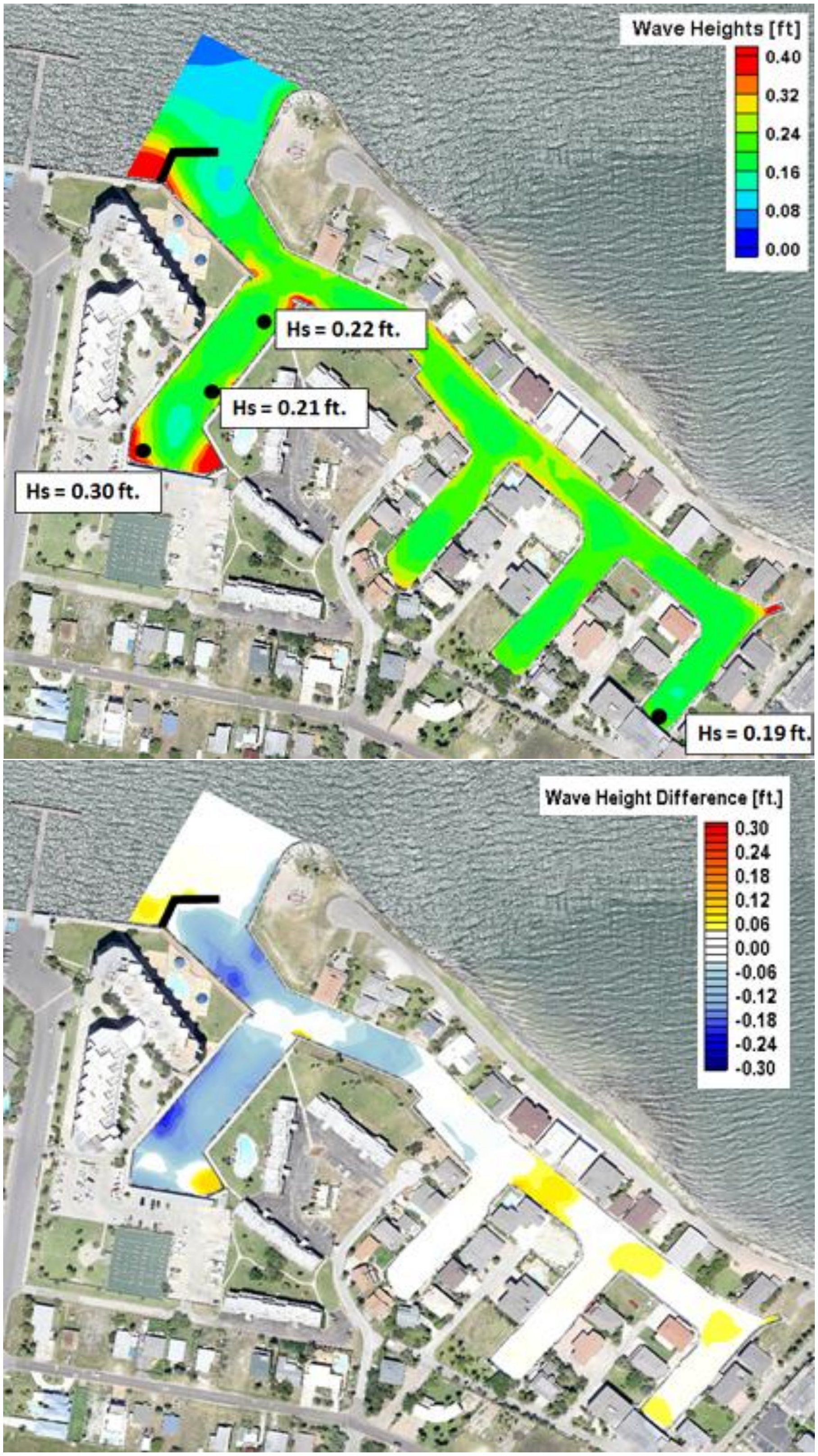

Figure B7. Significant breaking wave height (top) and significant breaking wave height difference from existing (bottom) for Alternative 6.

Appendix C

Maximum Velocity Plots

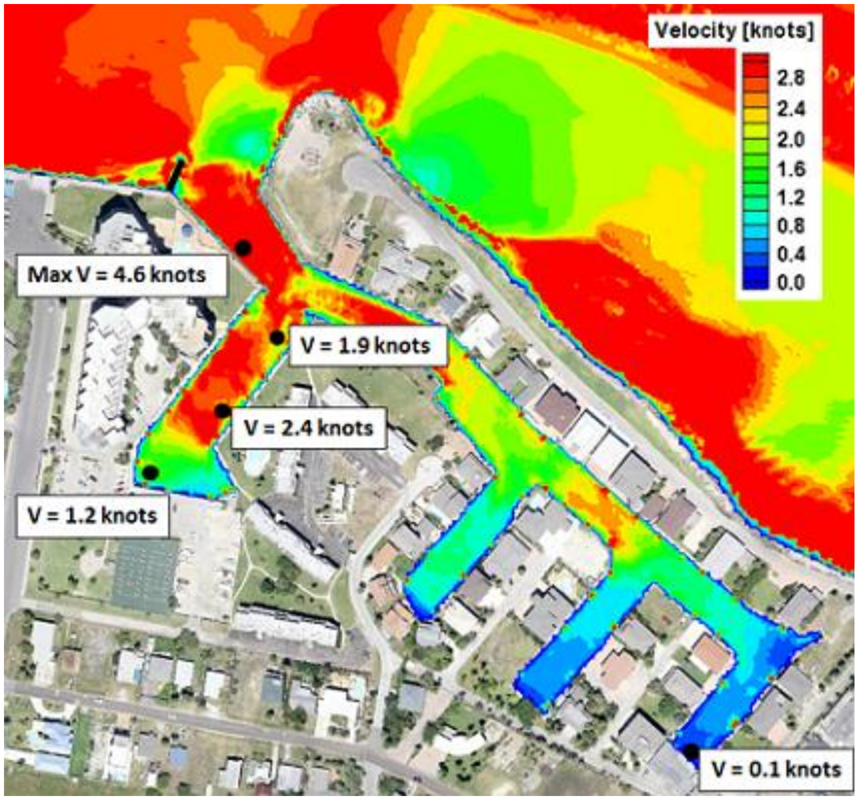

Figure C1. Maximum velocities throughout marina for Alternative 0 (existing conditions).

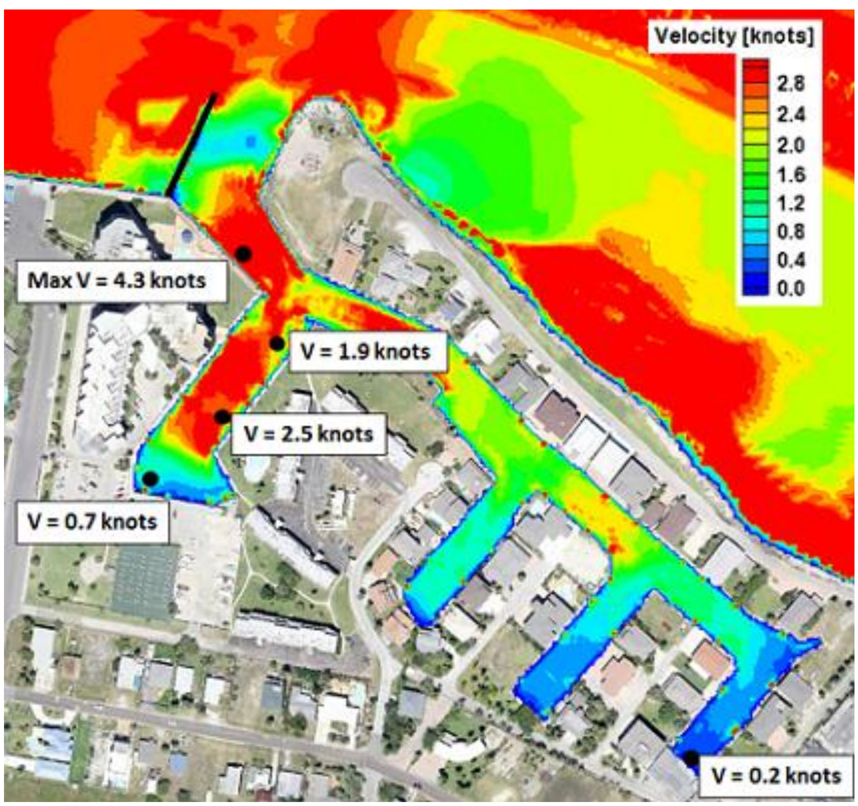

Figure C2. Maximum velocities throughout marina for Alternative 1.

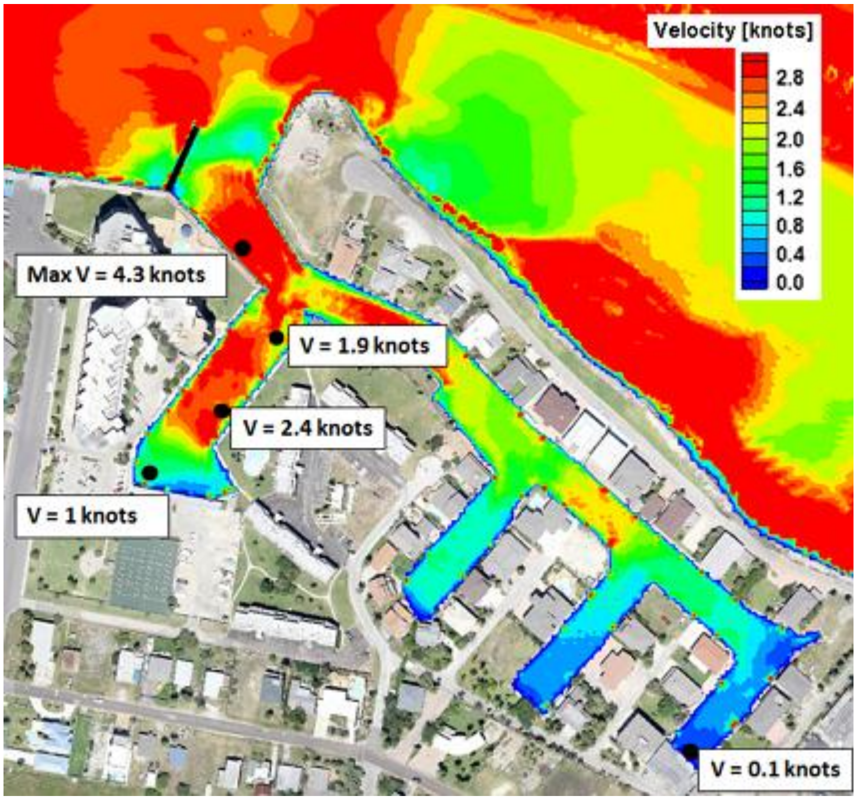

Figure C3. Maximum velocities throughout marina for Alternative 2.

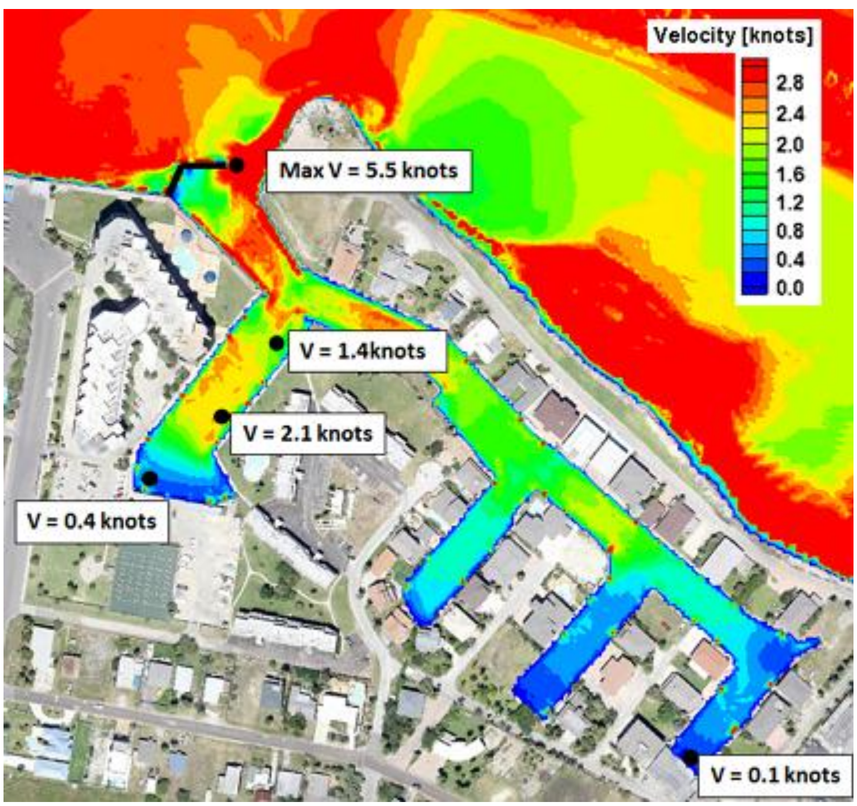

Figure C4. Maximum velocities throughout marina for Alternative 3.

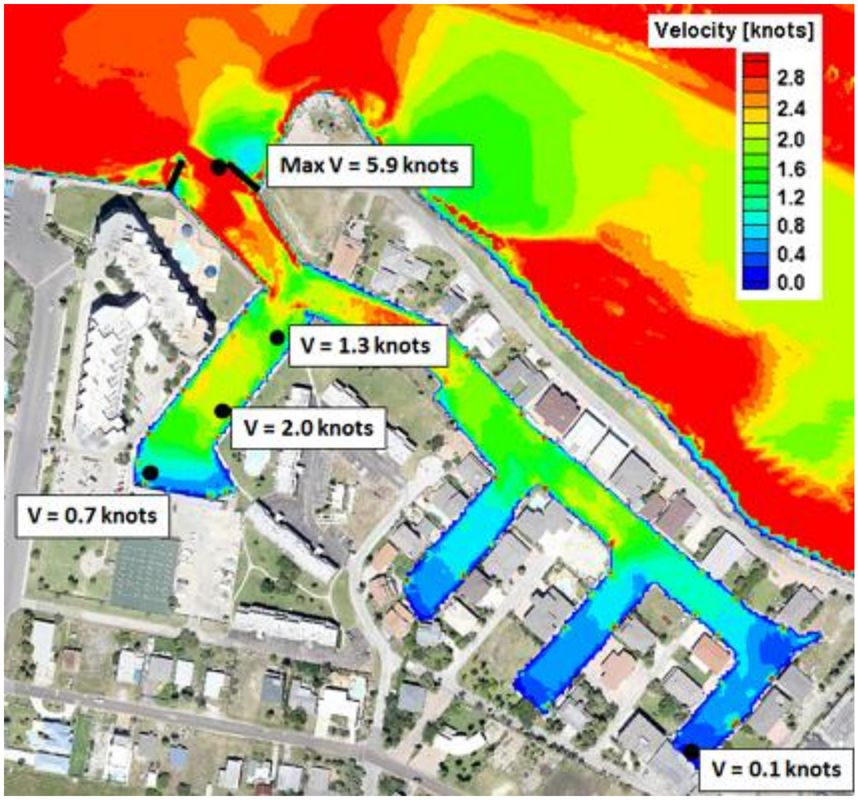

Figure C5. Maximum velocities throughout marina for Alternative 3A.

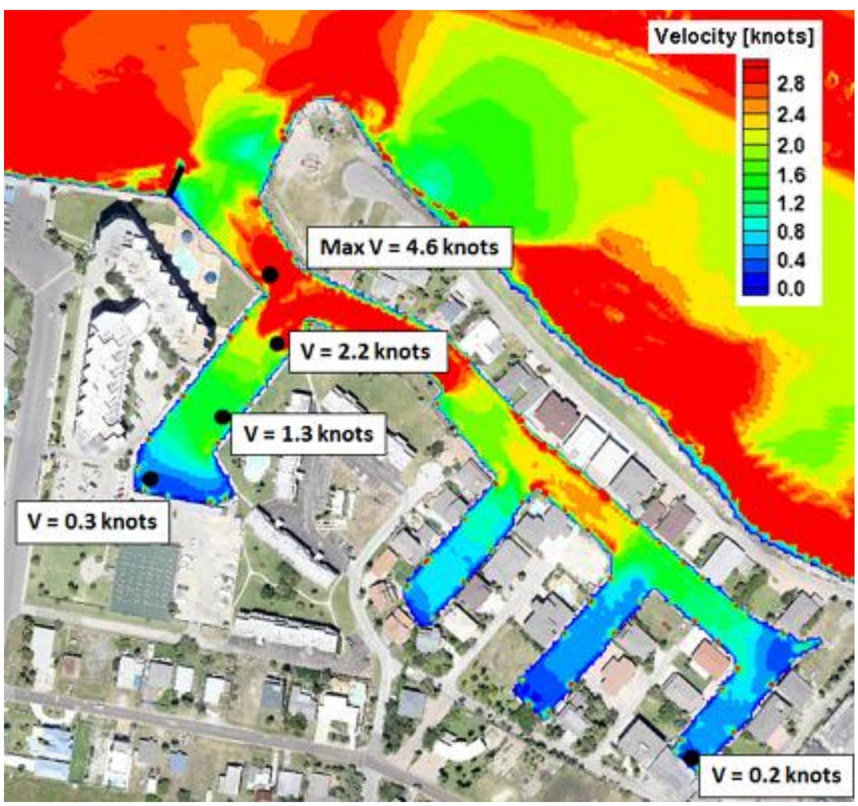

Figure C6. Maximum velocities throughout marina for Alternative 4.

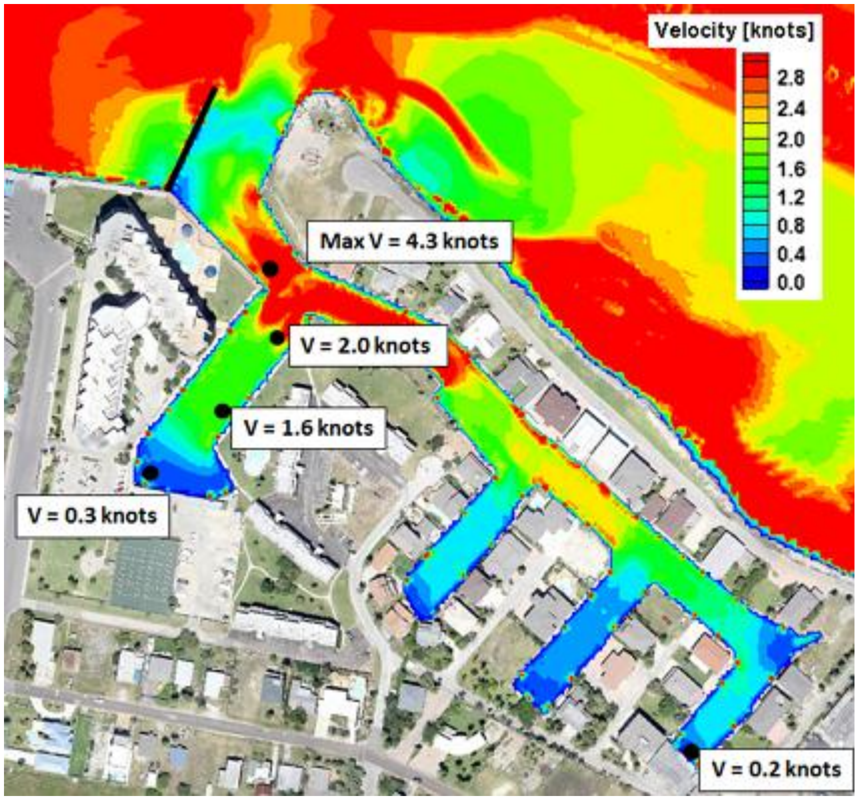

Figure C7. Maximum velocities throughout marina for Alternative 5.

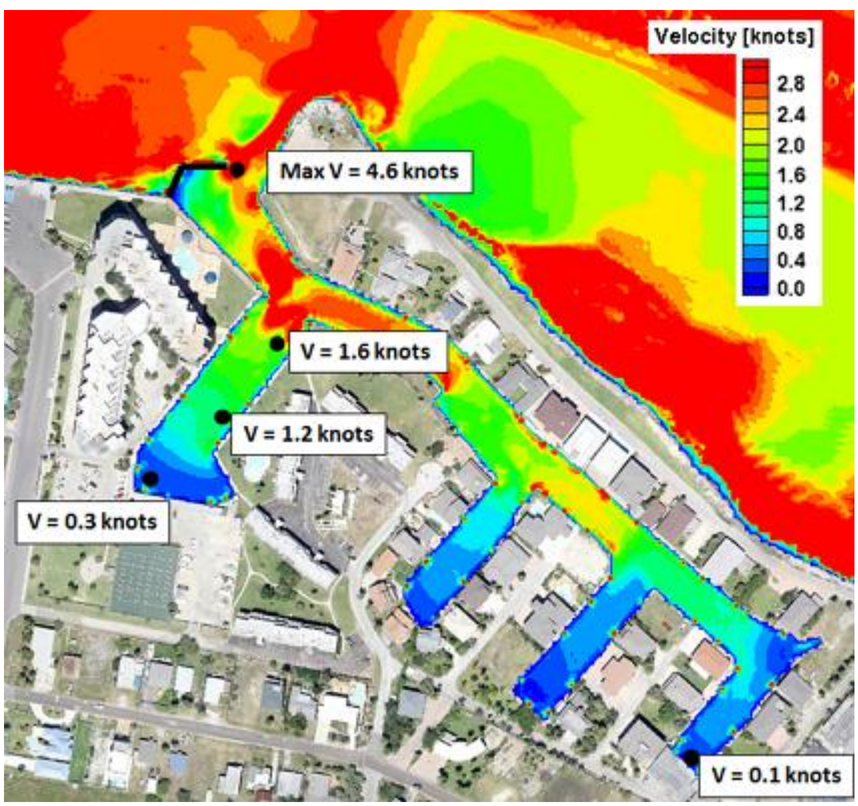

Figure C8. Maximum velocities throughout marina for Alternative 6.

Appendix D

Historical Aerial Photographs of the Project Site

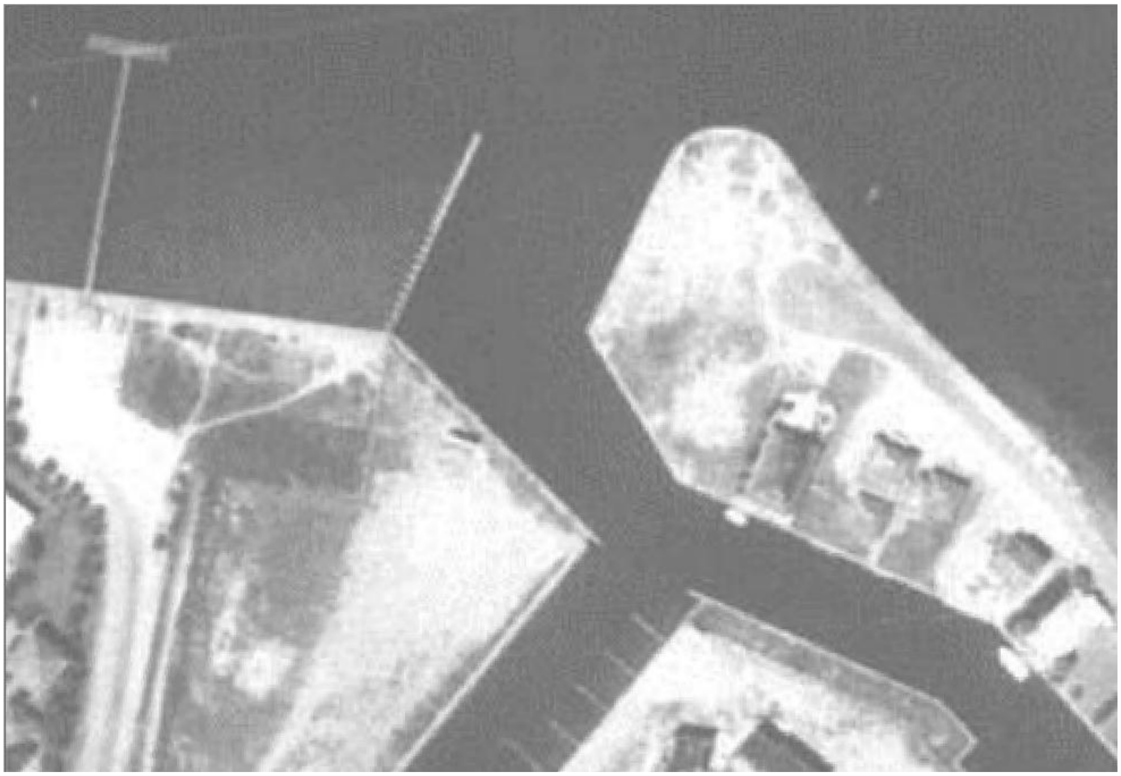

Figure D1. Aerial photograph of the project site in 1976 (breakwater length ~ 180 ft).

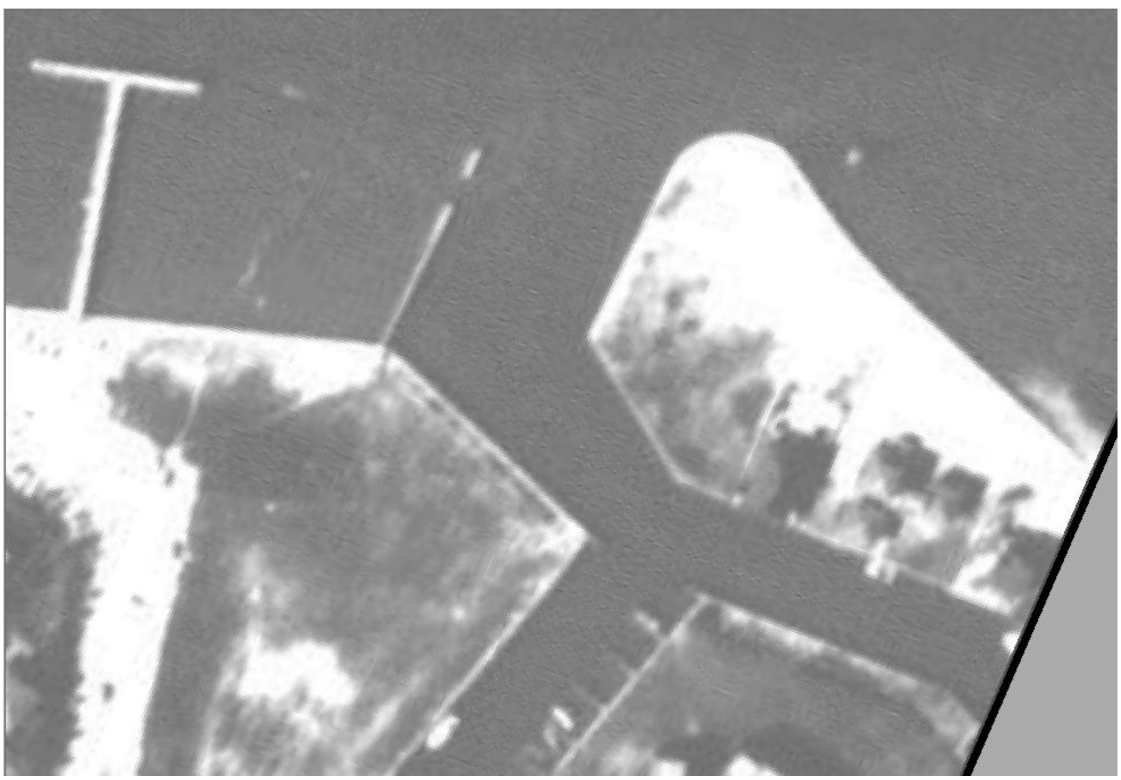

Figure D2. Aerial photograph of the project site in 1980 (breakwater length = 120 ft).

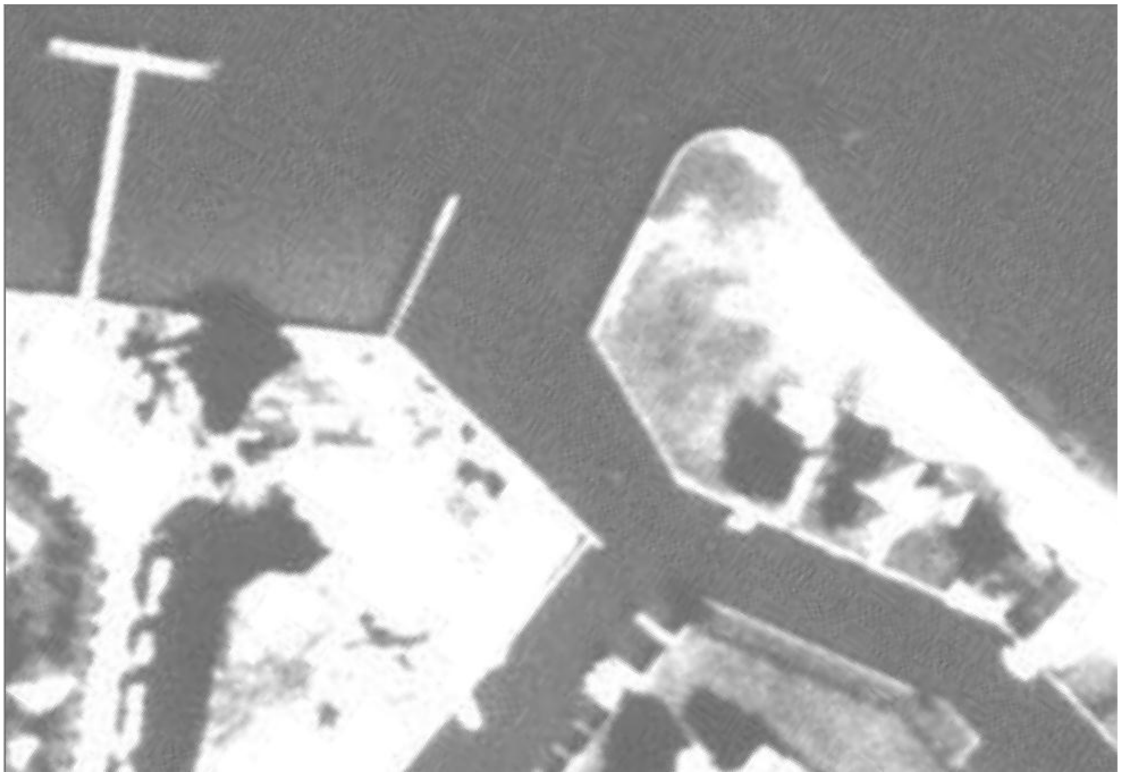

Figure D3. Aerial photograph of the project site in 1982 (breakwater length ~ 120 ft).

Figure D4. Aerial photograph of the project site in 1991 (breakwater length ~ 120 ft).

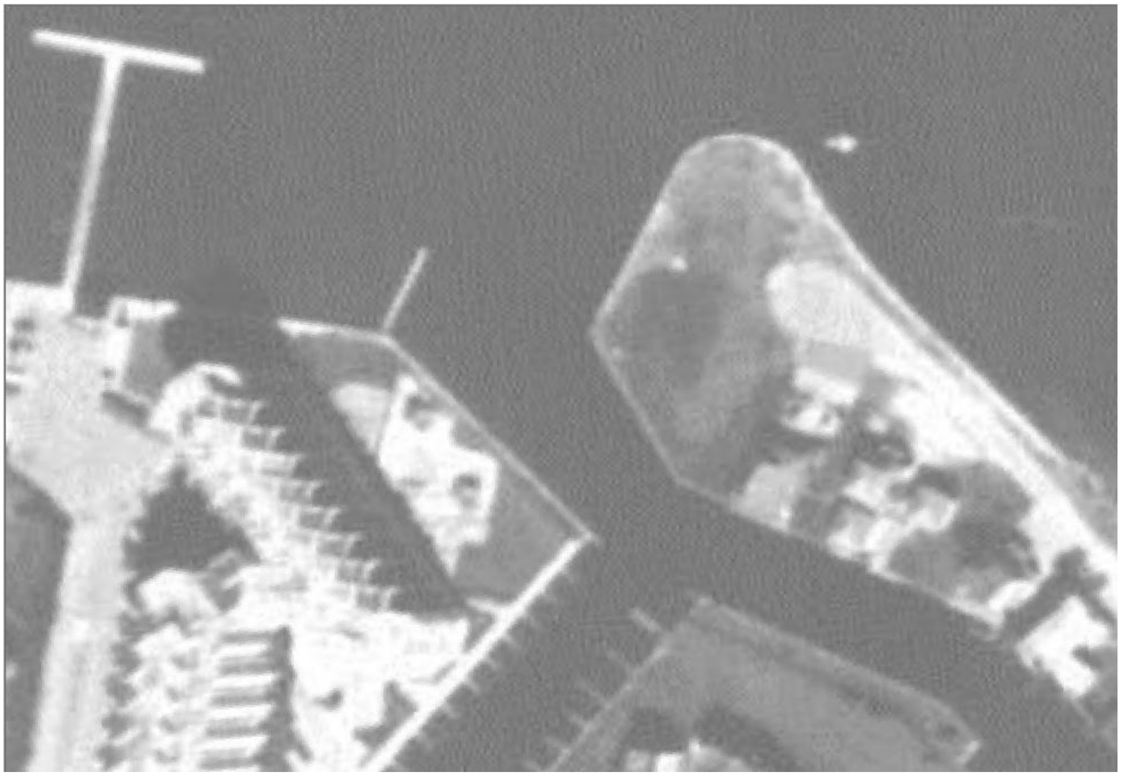

Figure D5. Aerial photograph of the project site in 1993 (breakwater length ~ 80 ft).

Figure D6. Aerial photograph of the project site in 2003 (breakwater length ~ 60 ft).

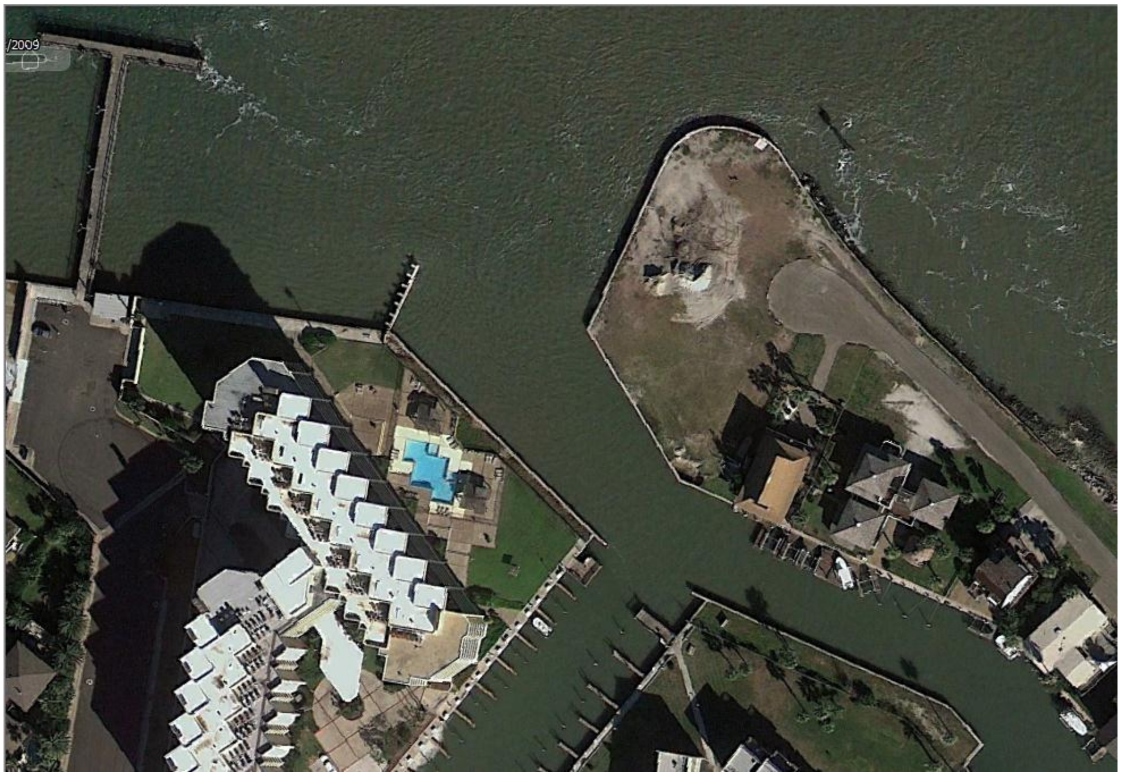

Figure D7. Aerial photograph of the project site in 2009 (breakwater length ~ 60 ft).

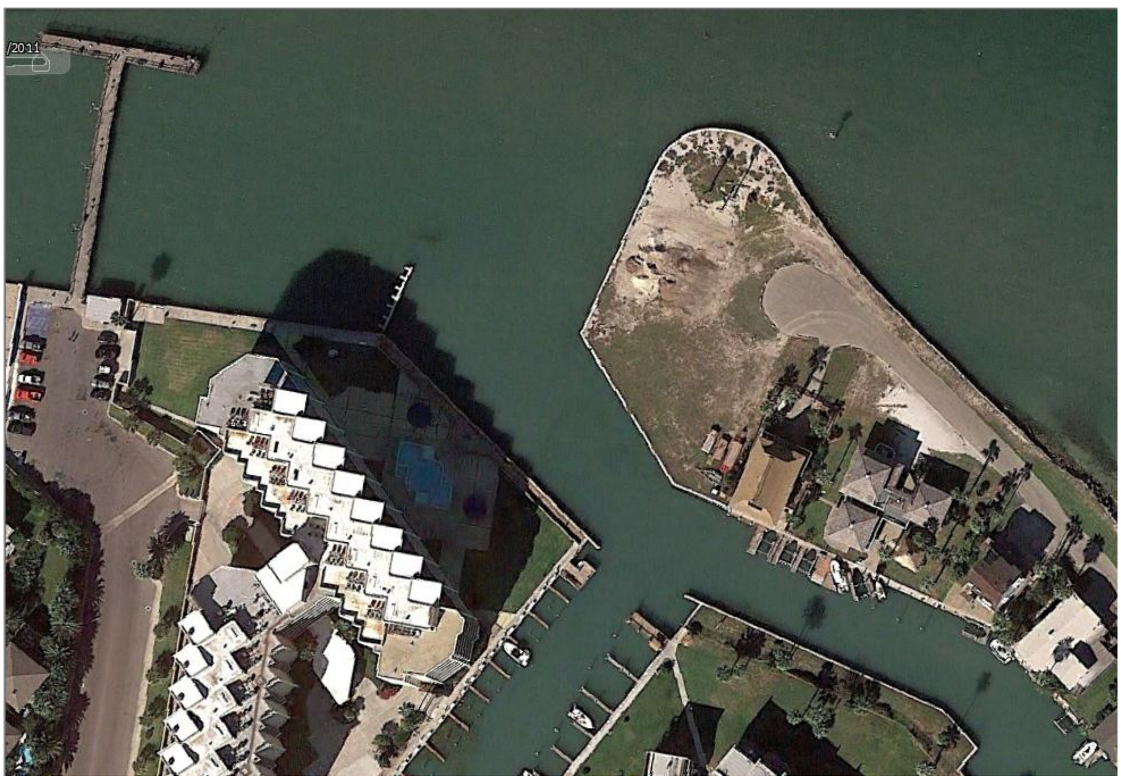

Figure D8. Aerial photograph of the project site in 2011 (breakwater length ~ 60 ft).

Figure D9. Aerial photograph of the project site in 2012 (breakwater length ~ 60 ft).

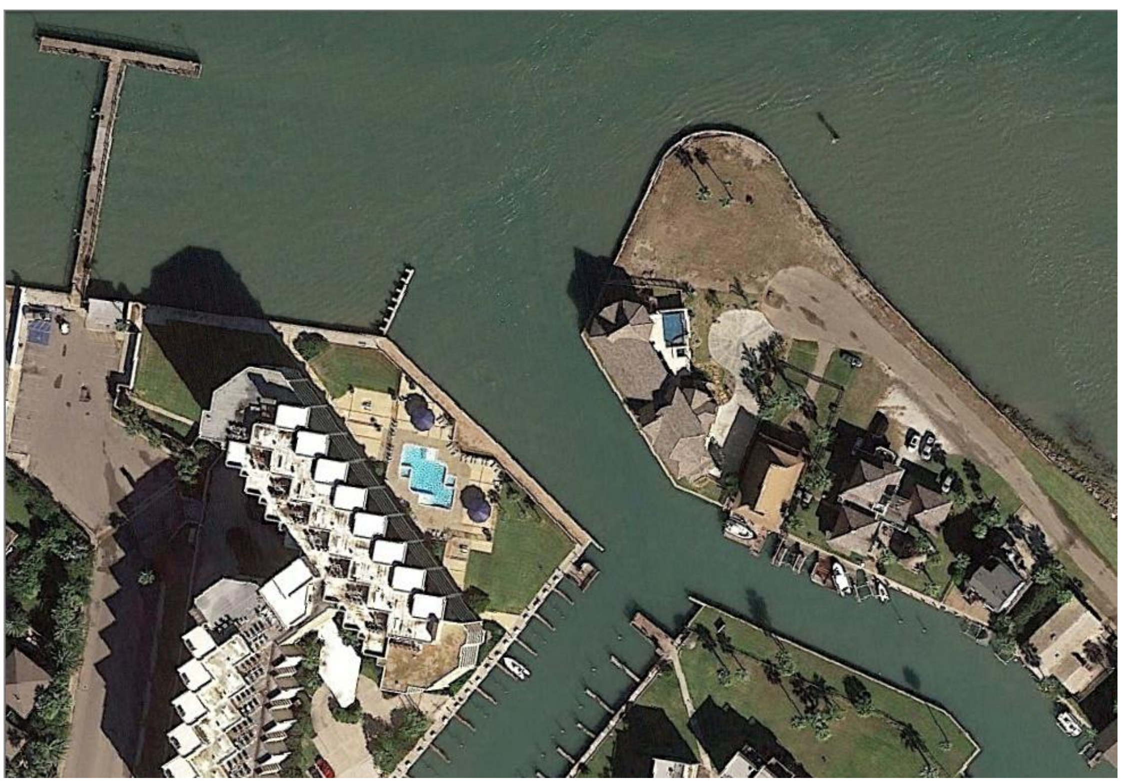

Figure D10. Aerial photograph of the project site in 2014 (breakwater length ~ 60 ft).