Coastal Impact - Modeling Study

Grand Port Site - Grand Bahama

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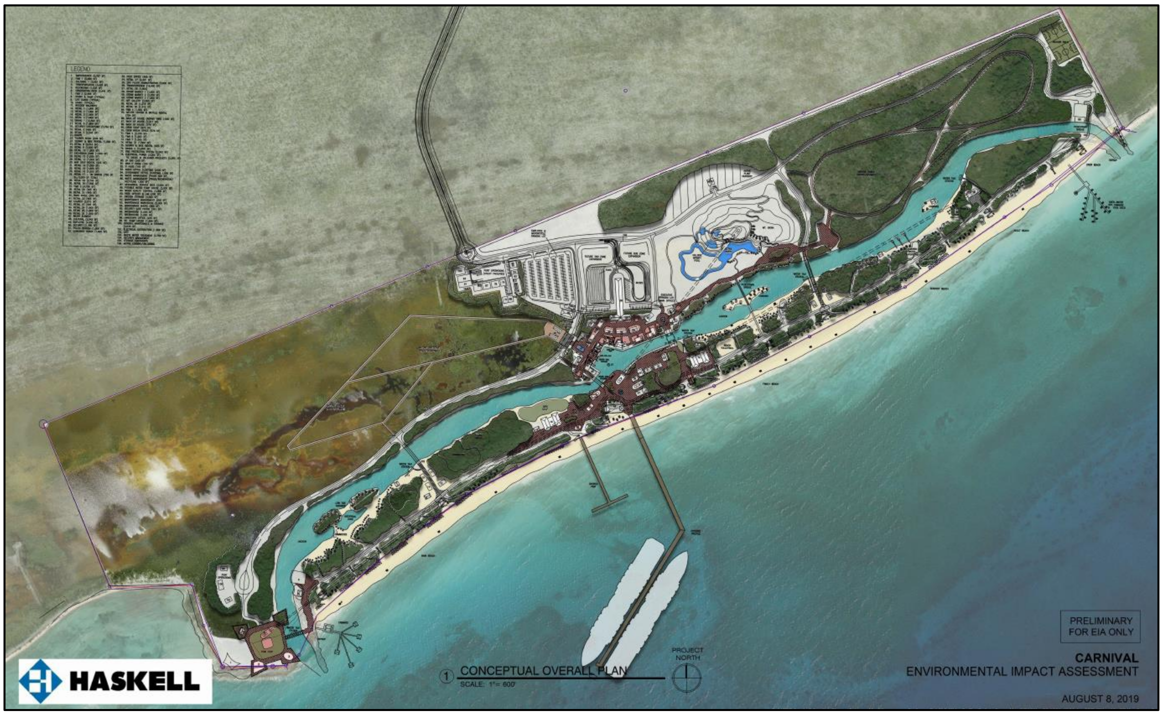
INTRODUCTION

This report summarizes the Task 3 coastal modeling analyses conducted by ATM to identify how the proposed Grand Port cruise ship project site will affect coastal conditions.

The primary environmental forces with the potential to affect the project site include wind, open ocean swell, wind-generated waves, currents, and water level changes including future sea level rise. The modeling analyses described herein provides an assessment of potential impacts to waves, currents, and sediment transport patterns as a result of the proposed project. Figure 1 presents the conceptual project layout for the EIA, used in this modeling analysis.



Setting Figure 1. Proposed Overall Plan (August 8, 2019)

The project site is situated on the southern shoreline of Grand Bahama, near the approximate center of the main island. The local shoreline faces the south-southeast towards the Northwest Providence Channel and is exposed to regionally generated deep-water waves over a range of southerly directions (Figure 2). The site is generally well protected from open ocean swell.

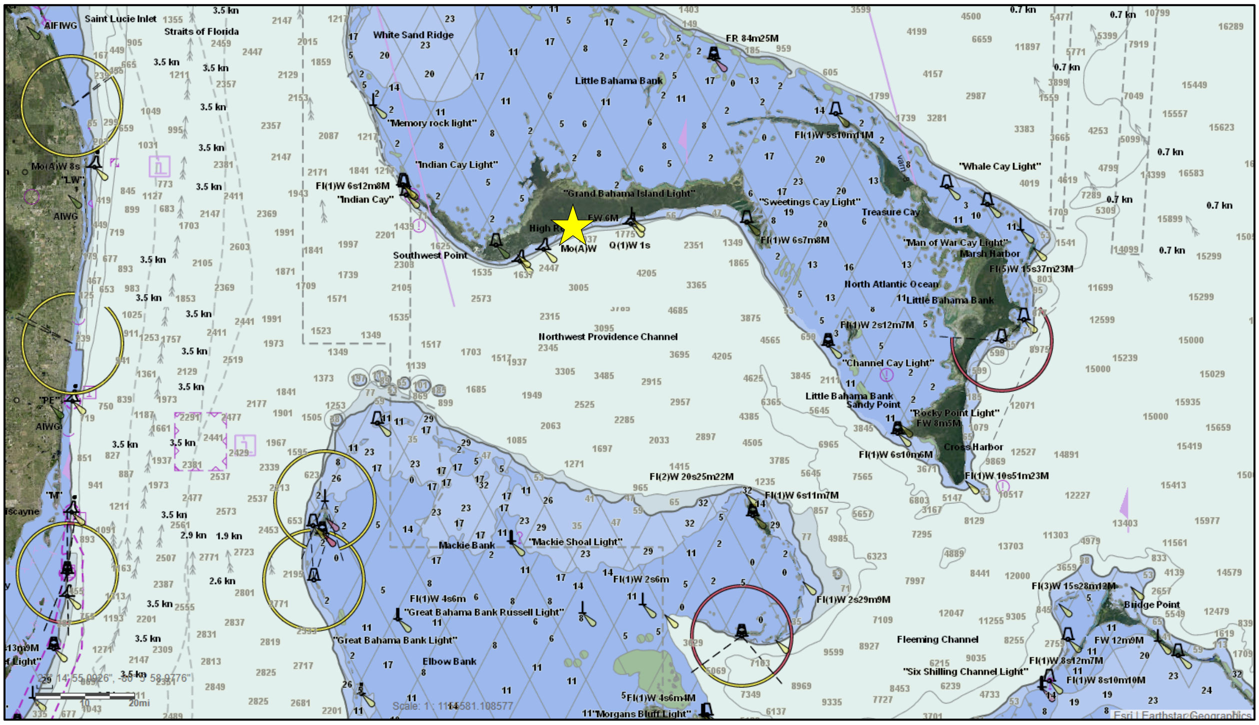


Figure 2. Project Location Exposure (source: NOAA Electronic Navigation Chart).

Local bathymetry data was collected at the project location and indicates nearshore depths sloping from the upland beach shoreline to approximately -35 ft below low water in the area of the proposed pier. This relatively shallow shelf extends to a depth of -90 ft MLW ~4000 ft offshore. Bathymetry drops off quickly from this shelf, with NOAA chart depths of over 600 ft approximately 5000 ft offshore. It should be noted that this modeling analysis was conducted prior to Hurricane Dorian and all bathymetric/topographic data used are representative of pre-storm conditions, and therefore are likely not representative of existing conditions today.

Previous studies by ATM and field data collections (by others) have been conducted to assess coastal conditions relevant to the project site, including a detailed analysis of measured currents, winds, and waves. Figure 2 presents the locations of site gauges deployed by Sea Diversified, Inc. to measure currents, waves, and winds directly at the project site.

These data, along with longer-term, regional datasets obtained from available sources were used for model development, calibration, and comparison to ensure model performance was realistically simulating conditions at the site.

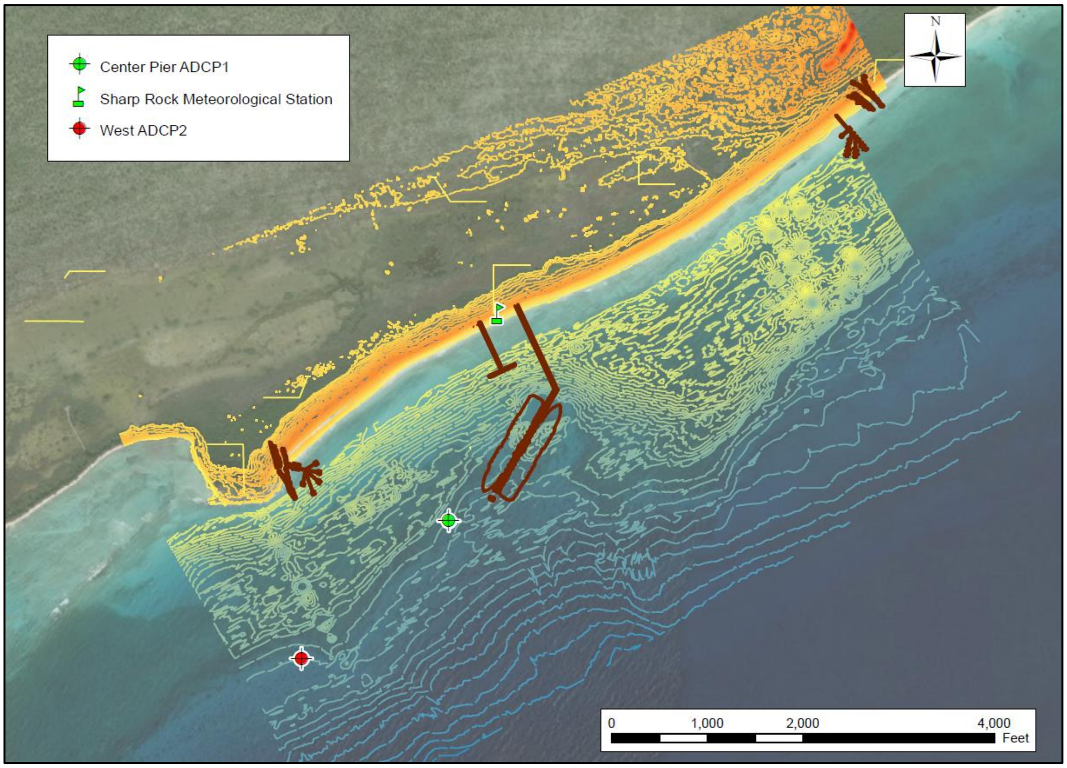


Figure 3. Project Site Location. Approximate proposed pier location, site bathymetry (by others), and ADCP/meteorological deployment locations by Sea Diversified, Inc. also shown.

MODELING ANALYSIS

A modeling study of the proposed project was conducted for long-term sediment transport analysis as well as to investigate potential changes in wave patterns (under typical and extreme wave conditions) and flushing potential of the planned canal. The Coastal Modeling System (CMS) was used for this application. The United States Army Corps of Engineers (USACE) developed CMS and it has been applied extensively in the United States and abroad. CMS is an integrated modeling system designed to simulate nearshore processes, especially with respect to navigation channel performance and sediment exchange between inlets and adjacent beaches. CMS couples flow, wave, and sediment transport models to simulate waves, current, water level, sediment transport, and morphology change.

CMS SETUP

Bathymetric and topographic data used in the study were primarily based on the local site survey and supplemented with bathymetric information obtained from local nautical charts (NOAA Chart #4149) and the NASA Shuttle Radar Topography Mission (SRTM) SRTM15 data set, which is a global data set featuring measurement on a 15-minute grid spacing.

All bathymetric and topographic data were converted to meters and referenced to the vertical datum of the Mean Sea Level (MSL), based on developed site tidal datums of measured water levels correlated to the nearest NOAA Station (Settlement Point Station: 9710441). A comprehensive merged dataset was then interpolated onto the model grid. Post-project conditions assumed a dredged canal depth of 7 ft below MLLW (~2.6 m MSL), and a dredged depth for the berthing area at the cruise ship pier of 35 ft below MLLW (~11.2 m MSL).

The CMS model suite (CMS-Wave and CMS-Flow) was used over the entire project region, out to deep water. Grid spacing for the CMS model domain is 4 meters. The CMS model grid domain and bathymetry are shown on Figure 4 (showing post-project input bathymetry conditions). CMS-Wave was coupled with CMS-Flow for long-term (30-day) model runs of currents and sediment transport of post-project conditions under representative/typical environmental conditions for the project area. Both offshore WaveWatch III (WW3) data and the deep-gauge ADCP wave measurements were used in developing representative wave condition inputs for the long-term model runs.

Additionally, "stand-alone" CMS-Wave model runs were conducted for extreme wave cases (established based on previous extreme value analyses of WW3 data) to assess pre and post-project impacts regarding potential changes in wave refraction, diffraction, and reflection behavior at the study location. More information is provided in the "Wave Impacts" section.

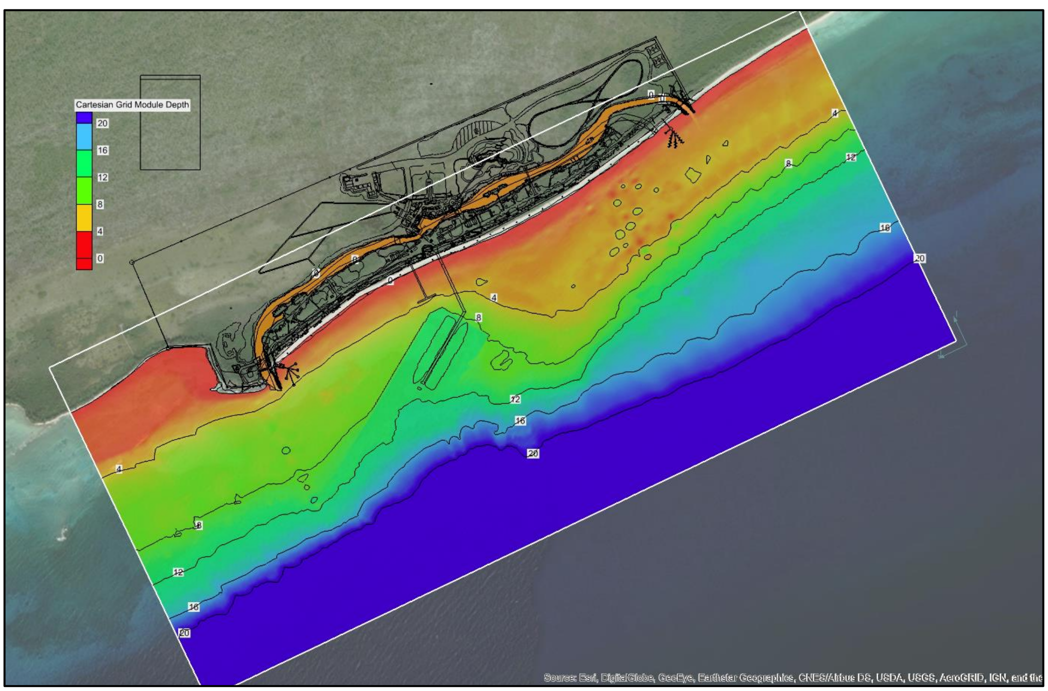


Figure 4. CMS Model Grid and Bathymetry Showing Post Project Conditions (Depths Shown are in Meters, MSL)

CMS-Wave, previously called WABED (Wave-Action Balance Equation Diffraction) was developed by the Coastal Inlets Research Program (CIRP) of the U.S. Army Corps of Engineering Research and Development Center, Coastal and Hydraulics Laboratory, in collaboration with two universities in Japan. CMS-Wave is a two-dimensional (2D) spectral wave transformation numerical model. It is designed for accurate and reliable representation of wave processes affecting operation and maintenance of coastal structures in navigation projects as well as in risk and reliability assessment of shipping in inlets and harbors. CMS-Wave is capable of simulating wave processes such as diffraction, refraction, reflection, wave breaking, and dissipation mechanisms, and the wave-current interaction.

The forward reflection coefficient was set to a constant 0.5 (default) in CMS-Wave. Forward reflection refers to obliquely reflected waves that are still traveling in the same general direction as the original wave. CMS-Wave solves the model grid by propagating waves in from the ocean boundary, then following this processing, it calculates backward reflection.

Backward reflection theoretically varies from 0 (no reflection) to 1 (complete reflection). For reference, it is noted that typically vertical sheetpile walls are assigned a 0.9 coefficient while rubble mound breakwaters are assigned ~0.5. Shorelines in the project area were given a backward reflection coefficient of 0.5 where rocky shorelines could easily be identified in aerial imagery, and elsewhere the default backward reflection coefficient of 0.3 was assigned. CMS-Wave was run with the default diffraction coefficient of 4, which corresponds to strong diffraction. Run-up was also included in all model runs.

In addition to waves, CMS-Flow is also driven by wind and tide inputs. These were developed for a representative 30-day run at the site using measured water level data from the ADCP deployments as well as windspeeds and directions measured at the Sharp Rock Met Station.

Flows and currents were evaluated and compared to other existing models and with measured data as available (i.e., ADCP measurements). As limited detailed sediment transport data is available for the site, the modeling effort did not include an extensive calibration effort of sediment transport, but rather aimed to simulate general known transport patterns for the area. Some sensitivity analysis was performed to qualify model response to parameters within the system and determine the most efficient and realistic model configuration. A spatial hard-bottom grid was created for the model domain based on aerial imagery and available information.

Currents and Sediment Transport

CMS was run under various conditions to assess currents at the site shoreline and surrounding areas, and particularly within the proposed canal/lagoon and inlets. Currents at the site are mostly mild (on the order of 0.1 m/s) and are known to be both tidally and wind-driven. Due to the small tide range here, tidally driven currents are small and can often be dominated by winds, which are predominantly out of easterly directions. As a result, site currents predominantly flow westward which drives longshore sediment transport in this direction as well.

Post-project currents and flushing potential of the proposed canal were conducted under two scenarios of 30-day runs for assessment:

- Scenario 1 – Tide Forcing Only
- Scenario 2 – Tide with Wind and Waves

Scenario 1 investigates a conservative case where conditions would be extremely calm, and only tidal exchange is driving currents and flow within the canal. The second scenario is more representative of typical site conditions where winds (primarily out of easterly directions) would be expected to drive currents in the canal the majority of the year.

Figures 5 and 6 present Scenario 1 model outputs of current velocities (in m/s) during a typical rising and falling tides, respectively, during the 30-day run. Figure 7 shows the maximum current magnitudes at any time over the 30-day run. This does not show a snap-shot in time like the previous figures, however, it is a useful for determining spatially how current velocities are distributed in order to gauge flushing potential for the proposed canal.

Under Scenario 1 (tide-only) conditions, current speeds in the lagoon are extremely slow, typically around 0.02 m/s. Relatively faster currents are observed at both inlets (around 0.05 m/s). As Figure 7 shows, there is an observed “nodal point”, where currents are observed to be consistently less than 0.01 m/s throughout the run. Based on the model outputs under this scenario, the canal would be unable to adequately flush due to tidal exchange alone.

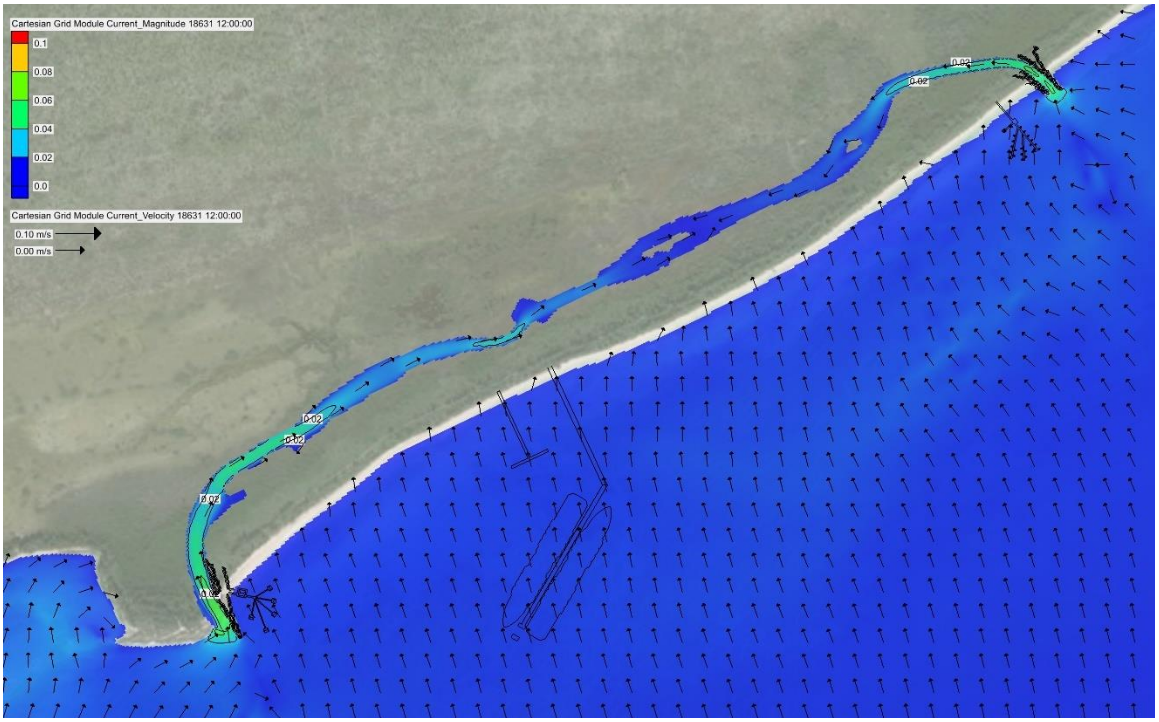


Figure 5. Scenario 1 (Tide Forcing Only) Current Velocity Outputs (m/s) During a Typical Rising Tide

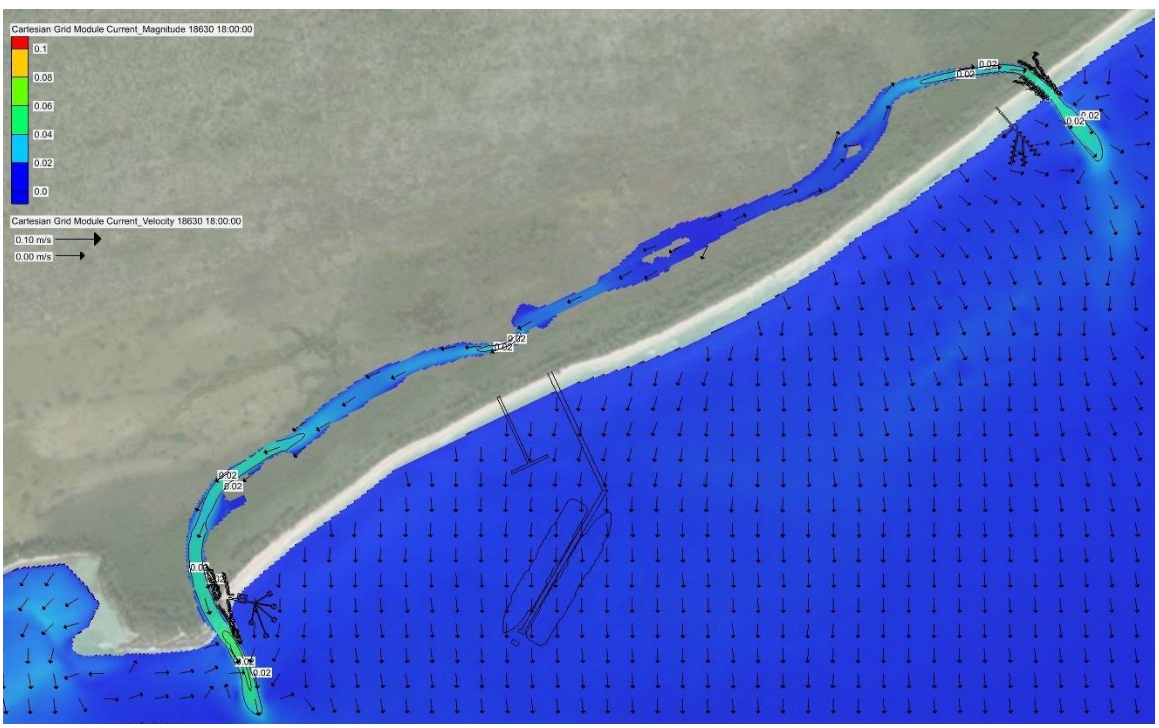


Figure 6. Scenario 1 (Tide Forcing Only) Current Velocity Outputs (m/s) During a Typical Falling Tide

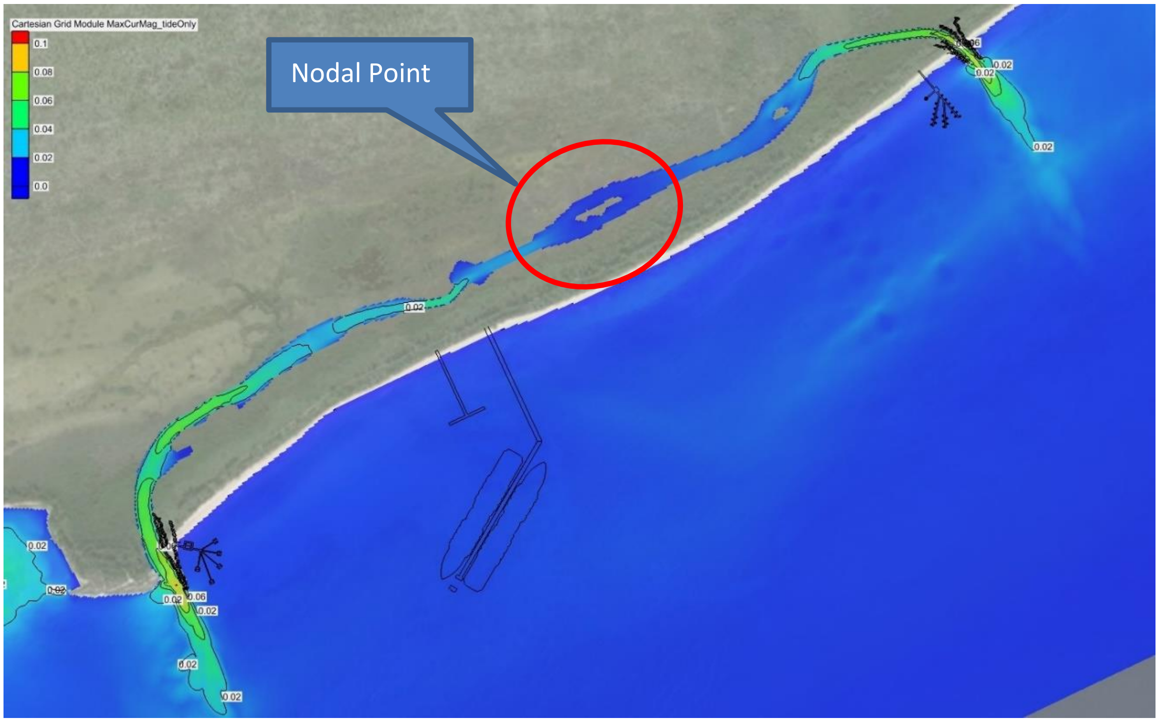


Figure 7. Scenario 1 (Tide Forcing Only) Maximum Current Magnitudes Over 30-Day Simulation

Similar to the Scenario 1 graphics analysis, Figures 8 and 9 on the following pages show typical rising and falling tide conditions for the Scenario 2 Condition where the CMS-Flow was run coupled with CMS-Wave and included wind speed and direction forcing. Note the change in scale on these figures from the Scenario 1 graphics, as winds and waves contribute a relatively significant amount to the flow. The maximum current magnitudes over the entire 30-day run of scenario 2 are shown on Figure 10. These represent a more realistic case of conditions representative at the site and compare well to measured values. Offshore currents are on the order of 0.1 m/s and increase moving closer to the shoreline, where they predominantly travel west-southwestward.

The Scenario 2 results provide further understanding of flushing potential for the proposed canal. Whereas, under the tide-only situation, currents would travel in and out both sides of the canal at least once every tidal cycle, this no longer occurs when typical site winds are included, which tend to dominate the flow direction in the canal. As a result, net flow would primarily occur from east to west under nominal conditions. During a rising tide, tidally driven flow opposes the predominant direction of wind-dominated flow causing a decrease in magnitude but no change in direction. The opposite happens during falling tides, and currents are increased moving from east to west within the lagoon.

However, as Figure 10 shows, maximum current speeds are consistently relatively small within the center of the canal/lagoon and particularly in the previously noted tidal nodal point of the canal. The waterway would still not meet water quality standards through natural flushing. This result is primarily attributable to the limited magnitude of local tidal forcing. Tide range in the area is limited and there is no phase lag in the tidal cycle between the two inlets. This results in very limited net flow (flushing) within the waterway. Modifications to the waterway design including the consideration of additional inlets would likely not result in substantial improvement to this condition given this tidal forcing mechanism.

Mechanical pumping is currently proposed to aid in flushing and increase water quality. Further analysis related to canal flushing will be conducted during design to determine the extent of mechanical pumping needs.

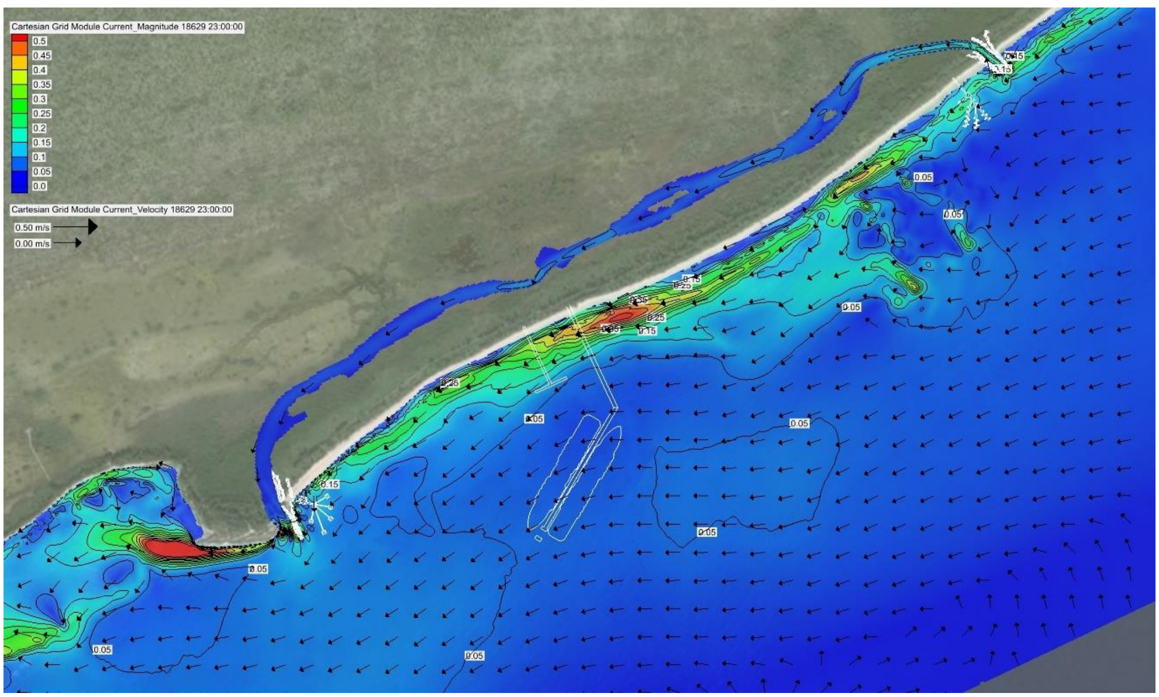


Figure 8. Scenario 2 Current Velocity Outputs (m/s) During a Typical Rising Tide

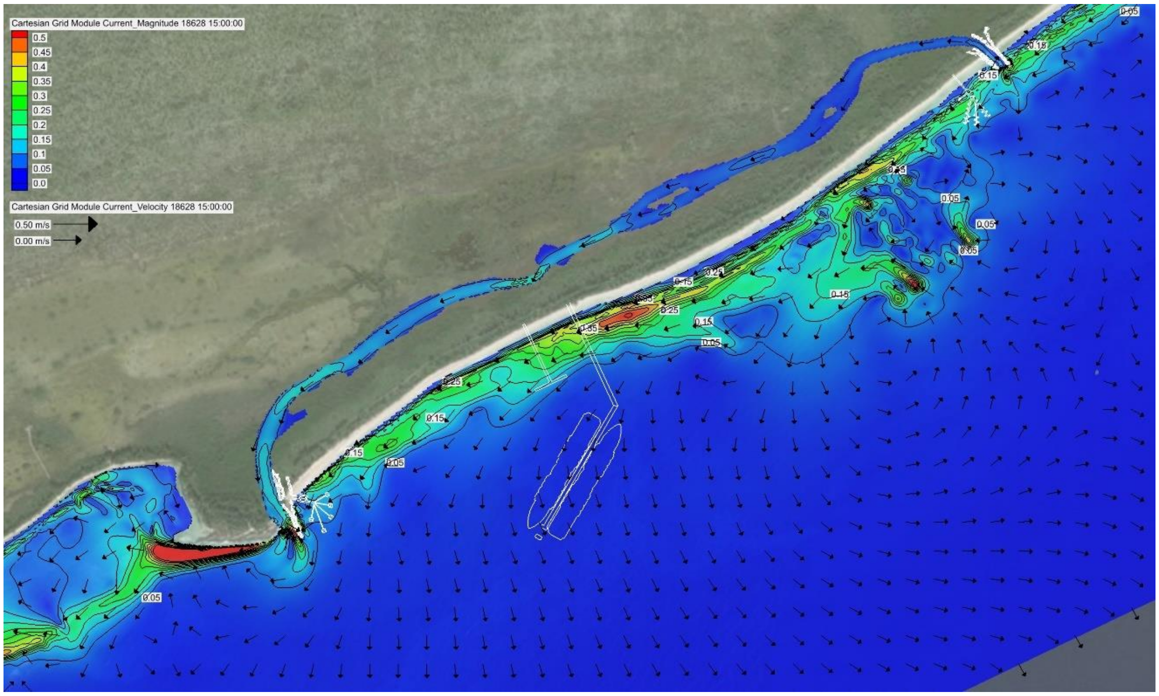


Figure 9. Scenario 2 Current Velocity Outputs (m/s) During a Typical Falling Tide

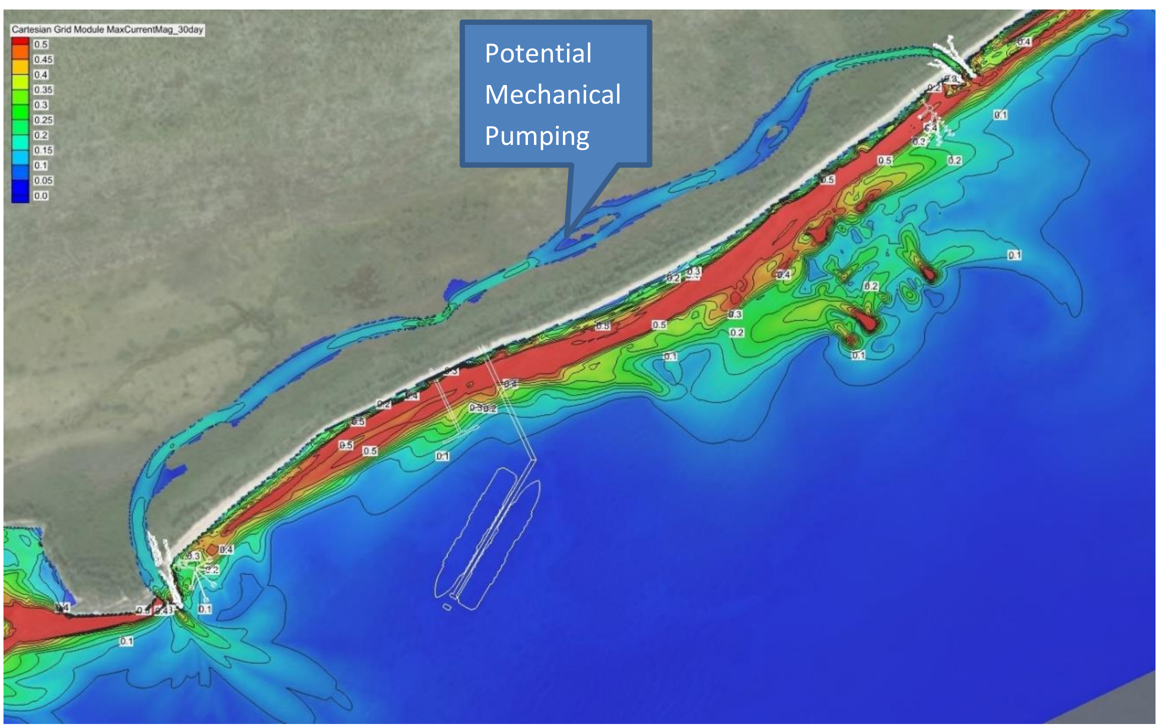


Figure 10. Scenario 2 Maximum Current Magnitudes Over 30-Day Simulation

In addition to currents and flushing potential, CMS was also used to assess sediment transport at the project site and nearby shorelines under post-project conditions. Figure 11 shows the model output of morphology change at the end of the month-long run of Scenario 2. Morphology change is a measure of exact depth/bathymetry change (in meters) over the simulation (yellow = accretion, blue = erosion). Areas shown as white represent locations where minimal or no changes occurred, and as expected Figure 11 shows the majority of sediment transport occurs near the shoreline.

Overall the observed morphology change shows transport directly at the project location is mostly cross-shore oriented, as areas of shoreline erosion generally see equal amounts of accretion into the nearshore. There is a long-shore transport component moving sand from northeast to southwest as well as evidenced by an accretion on the updrift (NE) side of the headland and proposed jetty at the southwestern inlet. In general, more significant erosion was shown to take place along the northeastern, updrift shoreline. The specific project site is within a littoral cell, due to the rocky headland. As a result, any down-drift impacts as a result of the proposed constructed jetty are expected to be minimal.

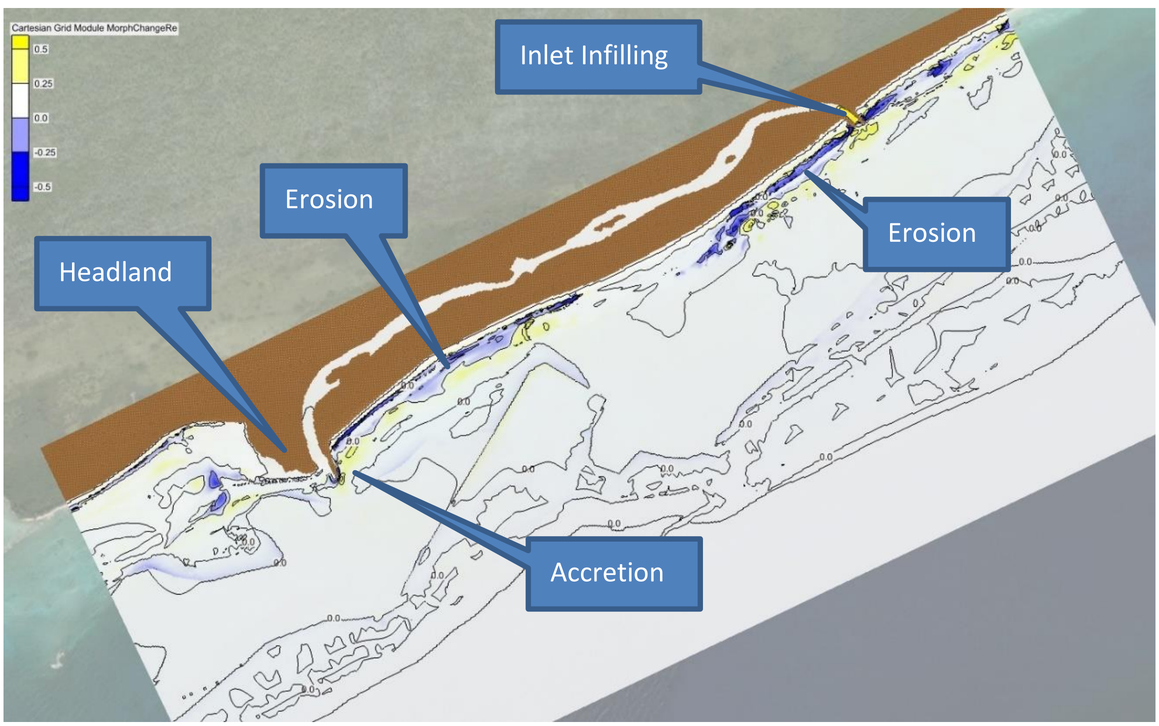


Figure 11. Scenario 2 - Morphology Change (in Meters) Following 30-Day Simulation

Transport patterns suggest some long-term erosion can be expected, particularly for the beach closer to the NE inlet. However, this can be maintained through nourishment efforts on an as-needed basis. Additionally, some inlet infilling is observed. The relatively high sedimentation rates of the inlet were observed mostly within the first few days of the 30-day run, and due mostly to equilibration of the cut channel and inlet, and model times later in the run showed sedimentation to slow over time Though rates following an equilibration period slowed, maintenance dredging of the inlets may be required on an as-needed basis, which could potentially provide beach-quality nourishment material.

WAVE IMPACTS

Wave impacts of pre- and post-project conditions were assessed for typical/operational and extreme wave conditions using the same CMS model domain described previously and running CMS-Wave model cases. Input model bathymetry of existing/pre-project and post-project conditions (post-project grid same as described previously and shown on Figure 4) are shown on Figure 12. The pre- and post- project conditions were modeled for various wave scenarios to assess potential changes in wave behavior as a result of the proposed project.

A CMS-Wave model input matrix was developed based on the offshore wave analysis and exposure. Table 1 presents the model input matrix used to run the varying simulations in CMS-Wave. Operational model runs were run at Mean Higher High Water (MHHW), and extreme model runs were based on 25- and 100-year conditions, where surge estimates were obtained from the Global Risk Assessment (GAR, 2015) for the site.

Table 1. "Stand Alone" CMS-Wave Model Simulations Matrix

Condition	*Case*	*Water Level (m, MSL)*	*Incident Offshore Wave Direction*	*Incident Offshore Hs (m)*	*Incident Offshore Tp (sec)*
Typical / Operational	**1**	0.5	SSE	1.2	6
	2		SW	1.2	6
	3		ESE	1.2	6
Extreme 25-Yr	**4**	2.2	SSE	6.3	10
	5		SW	6.3	10
	6		ESE	6.3	10
Extreme 100-Yr	**7**	2.5	SSE	9.3	12
	8		SW	9.6	12
	9		ESE	9.3	12

CMS-Wave model output stations were developed to quantitatively analyze wave heights at the site and potential increases/decreases following berth dredging and construction of inlet stabilization structures (i.e., jetties). Output station locations are shown on Figure 13, relative to the proposed site layout.

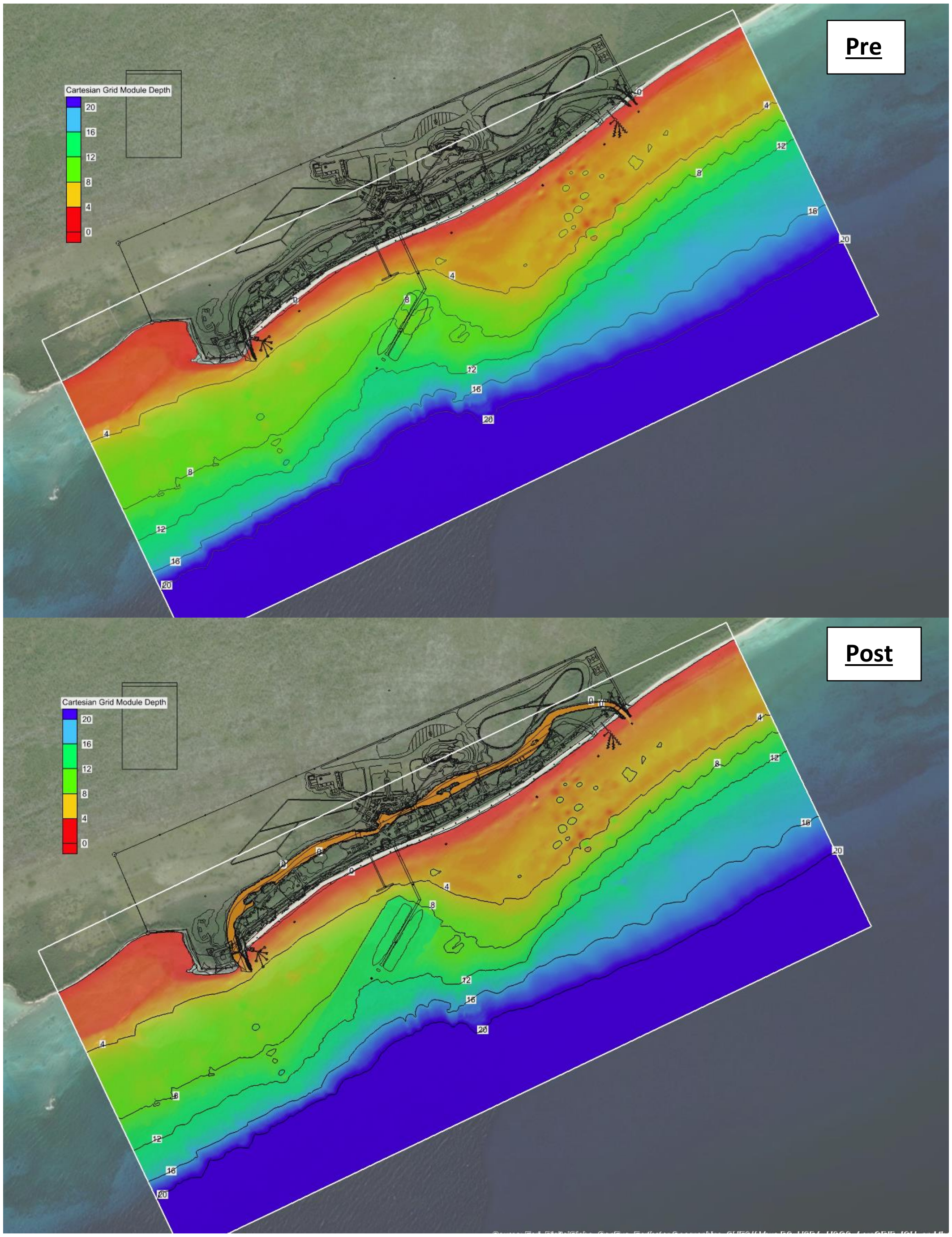


Figure 12. (Upper) Pre-Project and (Lower) Post-Project Bathymetry. Site Layout Shown on Pre-Project Grid Only for Reference. Depths in Meters Referenced to MSL.

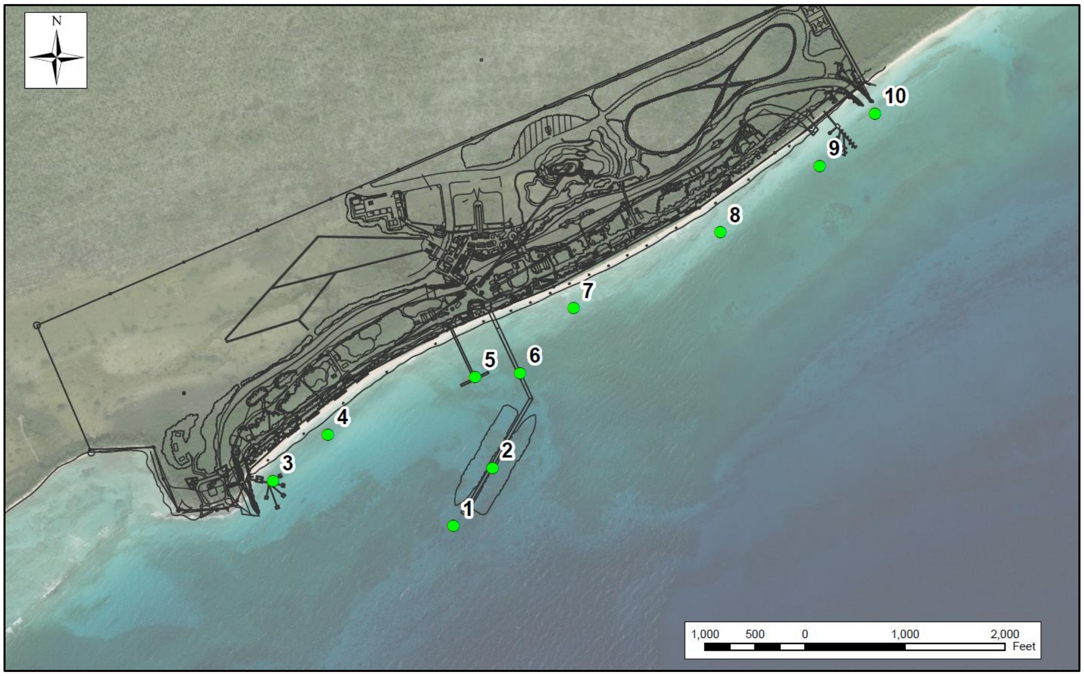
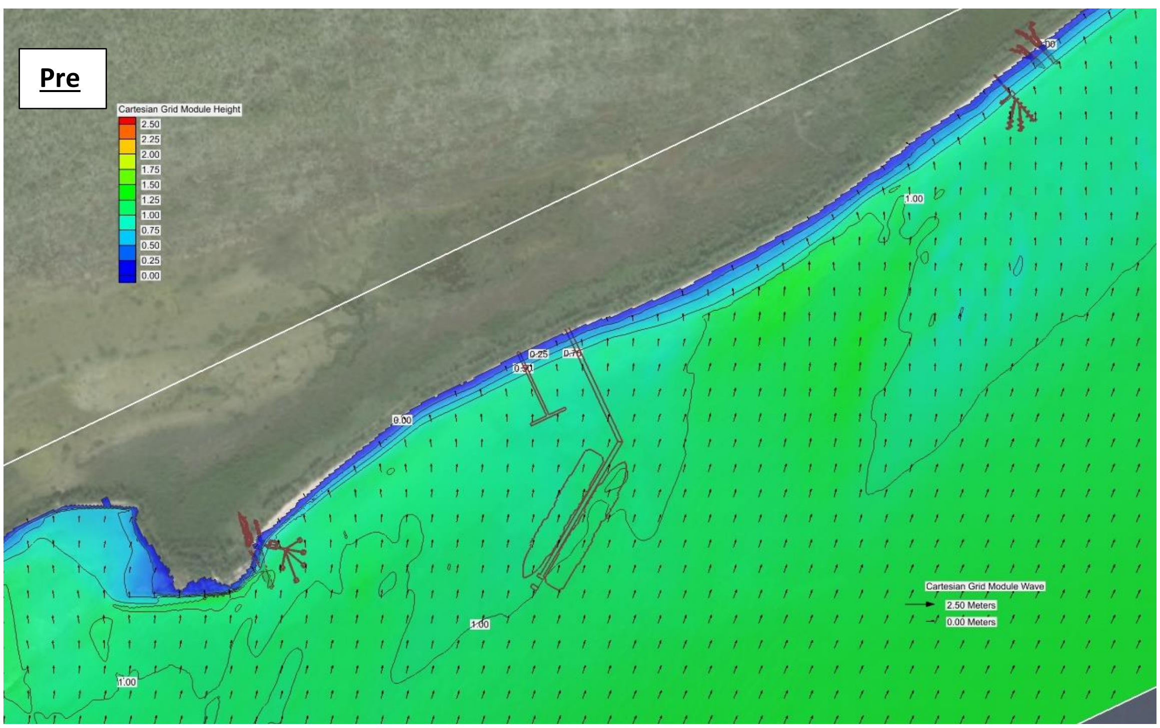


Figure 13. CMS-Wave Model Output Station Locations

For a direct quantitative comparison, Table 2 below presents the percent change in wave heights between pre- and post- project conditions at the model output stations. Positive values indicate wave height increases under post-project conditions and negative values indicate decreases. Tables showing model all output station wave heights are provided in the report appendix. Figure 14 presents the graphical model outputs for visual comparison of pre- and post-project wave heights and direction for Case 2. Graphical model all nine (9) wave cases are provided in the appendix. Note that all wave heights are in meters and that project site plans are shown on existing condition graphics only for reference.

Table 2. Percent Change in Wave Heights at Model Output Stations (Post Minus Pre)

Output Station	Case 1 Typical SSE	Case 2 Typical SW	Case 3 Typical ESE	Case 4 25-Yr SSE	Case 5 25-Yr SW	Case 6 25-Yr ESE	Case 7 100- Yr SSE	Case 8 100- Yr SW	Case 9 100- Yr ESE
1	7%	5%	6%	0%	1%	0%	0%	0%	0%
2	7%	8%	6%	2%	5%	0%	5%	8%	2%
3	5%	-3%	5%	0%	-2%	1%	0%	0%	0%
4	0%	1%	2%	0%	0%	0%	0%	0%	0%
5	-9%	-7%	-8%	0%	-1%	-4%	0%	6%	0%
6	-1%	2%	4%	0%	2%	3%	0%	3%	5%
7	0%	0%	0%	0%	0%	0%	0%	0%	0%
8	0%	1%	0%	1%	0%	1%	1%	0%	1%
9	2%	2%	2%	0%	0%	0%	0%	0%	0%
10	4%	5%	2%	0%	0%	0%	0%	0%	0%



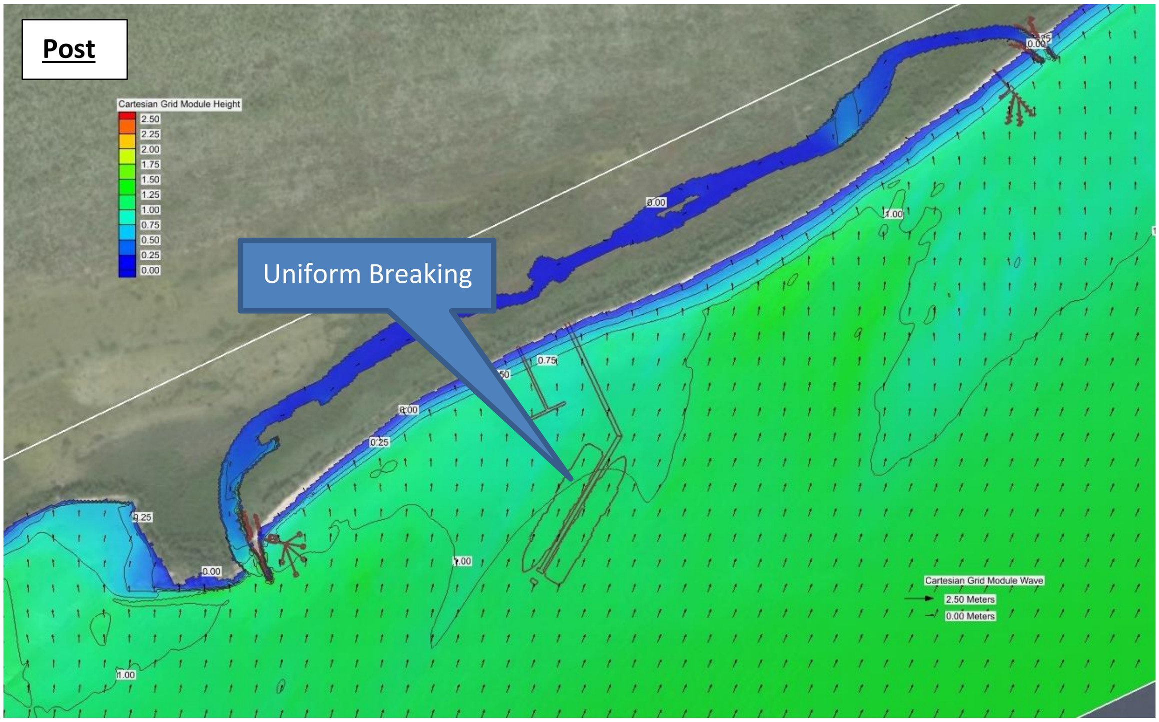


Figure 14. Case 2 (Typical SW Offshroe Incident Wave) CMS-Wave Model Height Outputs of (Upper Panel) Pre – and (Lower Panel) Post-Project Conditions. Note that project site plan elements (piers, jetties, overwater bungalows) are shown on the pre-project graphic (Upper Panel) only for reference.

As Table 2 and Figure 14 show, anticipated changes in wave heights as a result of the project are minimal (less than 10%). Due to the berth dredging at the pier location, relatively larger waves are able to propagate and break closer to the shoreline. Any increased erosion as a result of waves may cause some increased erosion in these areas, and may require nourishment efforts following large storms.

Figure 15 shows the post-project condition of Case 1, to highlight that some reflection off of the southeast jetty could occur under both typical and extreme conditions and should be planned for during design. The wave heights observed here could be decreased through mild construction slopes and the use of less reflective construction materials.

In general, however, changes in breaking, refraction, and reflection in model outputs as a result of the project are expected to be minor and any potential impacts are confined to the specific project location

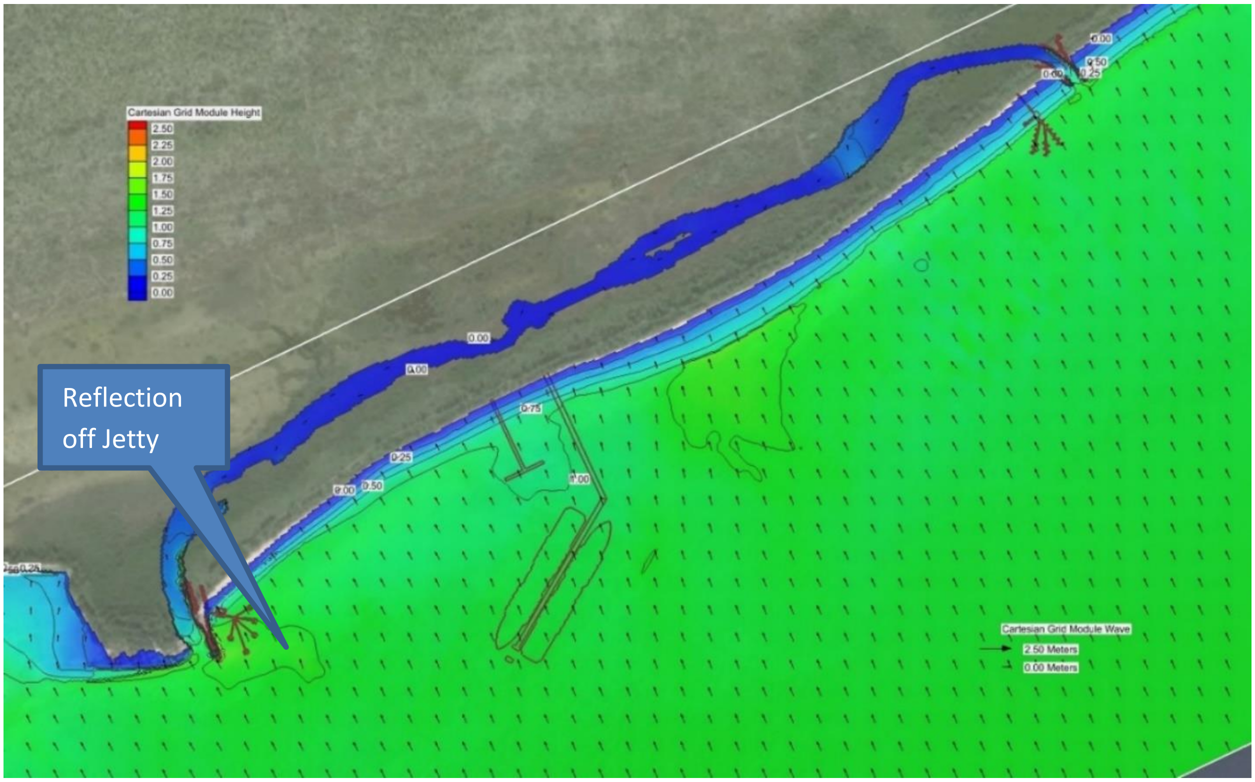


Figure 15. Post-Project Wave Height Outputs (Case 1, Typical SSE Incident Wave)

SUMMARY AND CONCLUSIONS

This report summarizes the modeling analyses conducted by ATM to identify potential impacts as a result of the proposed facility at the Grand Port – Grand Bahama project site. Based on the modeling effort and analysis, project impacts can be generally characterized by the following:

- The proposed canal will likely not meet water quality standards due to its inability to flush naturally unless mechanical pumping is implemented.
- Mechanical pumping is proposed, and the design and required system will be determined following further analysis during the design phase.
- Changes in beach stability (erosion potential) will be primarily limited to interior portions of the property.
- Infilling of the eastern inlet channel is predicted suggesting that maintenance dredging may be required. This material can be beneficially used to address erosion to the immediate west of the inlet.
- The main areas of increased erosion potential occur adjacent and downdrift (to the west) of the proposed eastern canal inlet and immediately landward of the berth.
- Downdrift impacts as a result of the project outside of the property boundaries will be minimal as the Sharp Rock headland already largely isolates the project site within its own littoral cell.
- Dredging of the berth will cause waves to refract and break closer to shore. These impacts in terms of increased wave heights at the shoreline and erosion potential are minor and limited to the study area.
- Increased wave energy seaward of the proposed jetties is possible due to wave reflection. This can be mitigated during design through mild sloping structures and/or the use of less reflective material.

APPENDIX : CMS-WAVE OUTPUTS

Site Plan Layout Shown on all Pre-Project Graphics for Reference Only

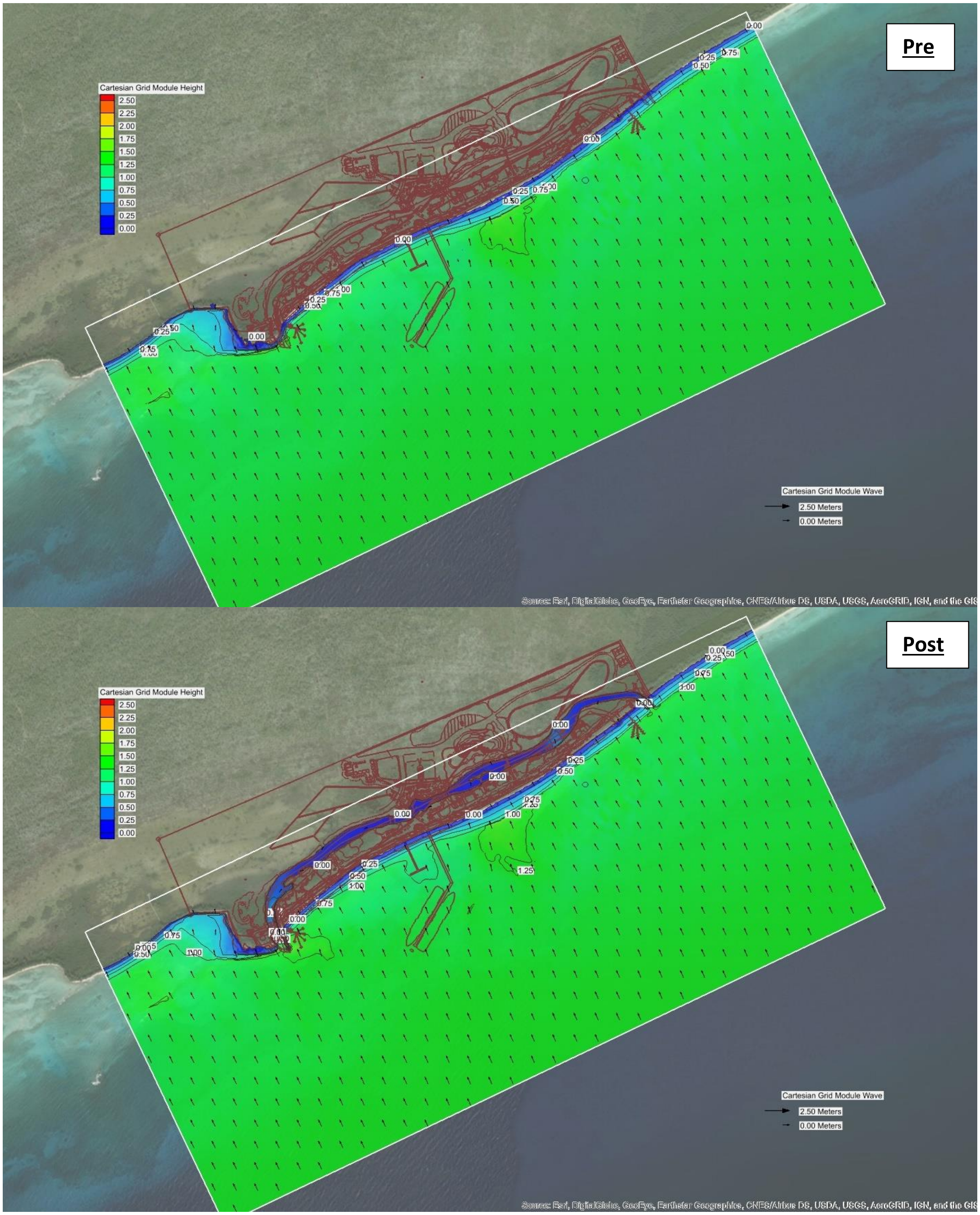


Figure 16. Case – 1 (Typical SSE) Wave Height and Direction Outputs

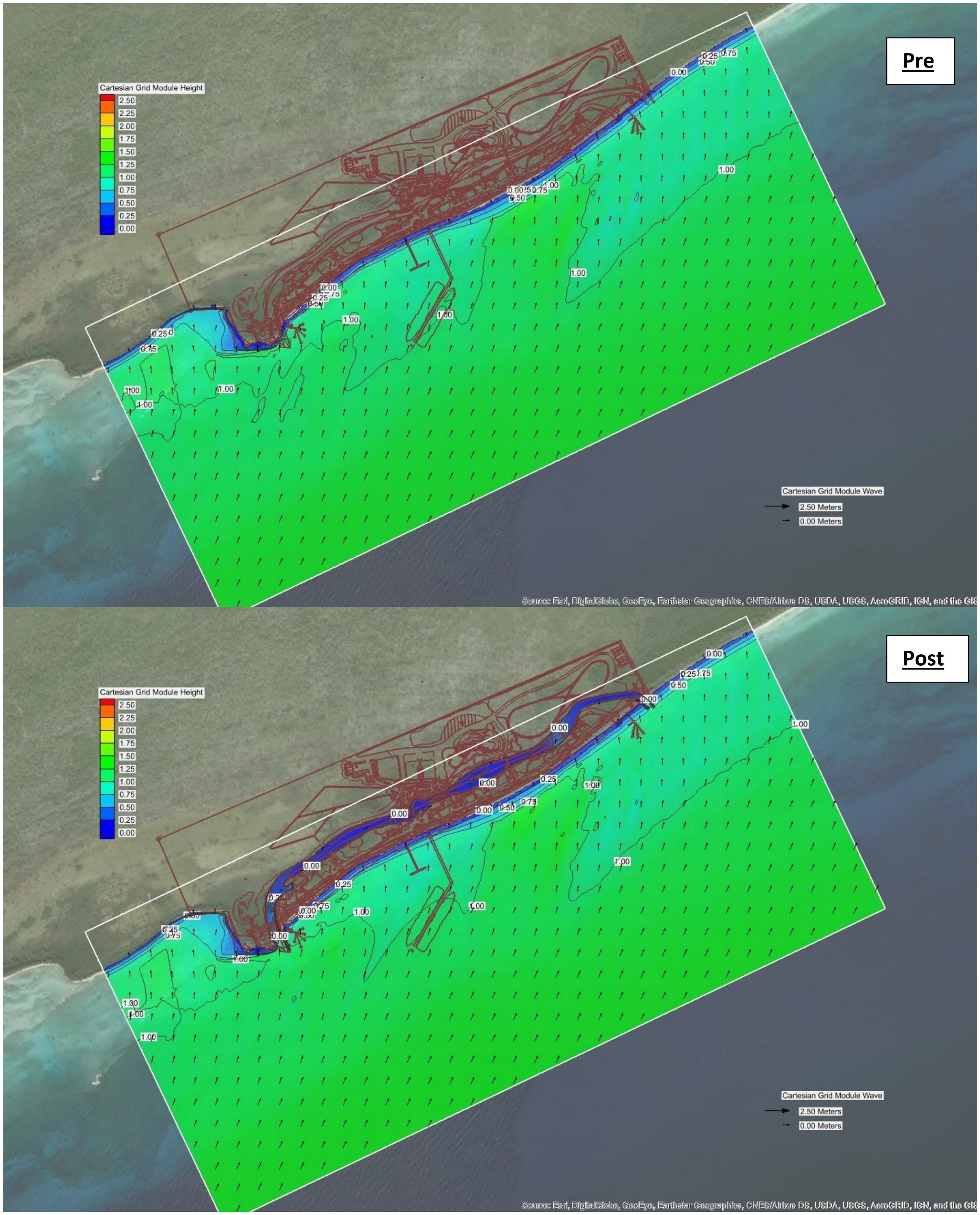


Figure 17. Case – 2 (Typical SW) Wave Height and Direction Outputs

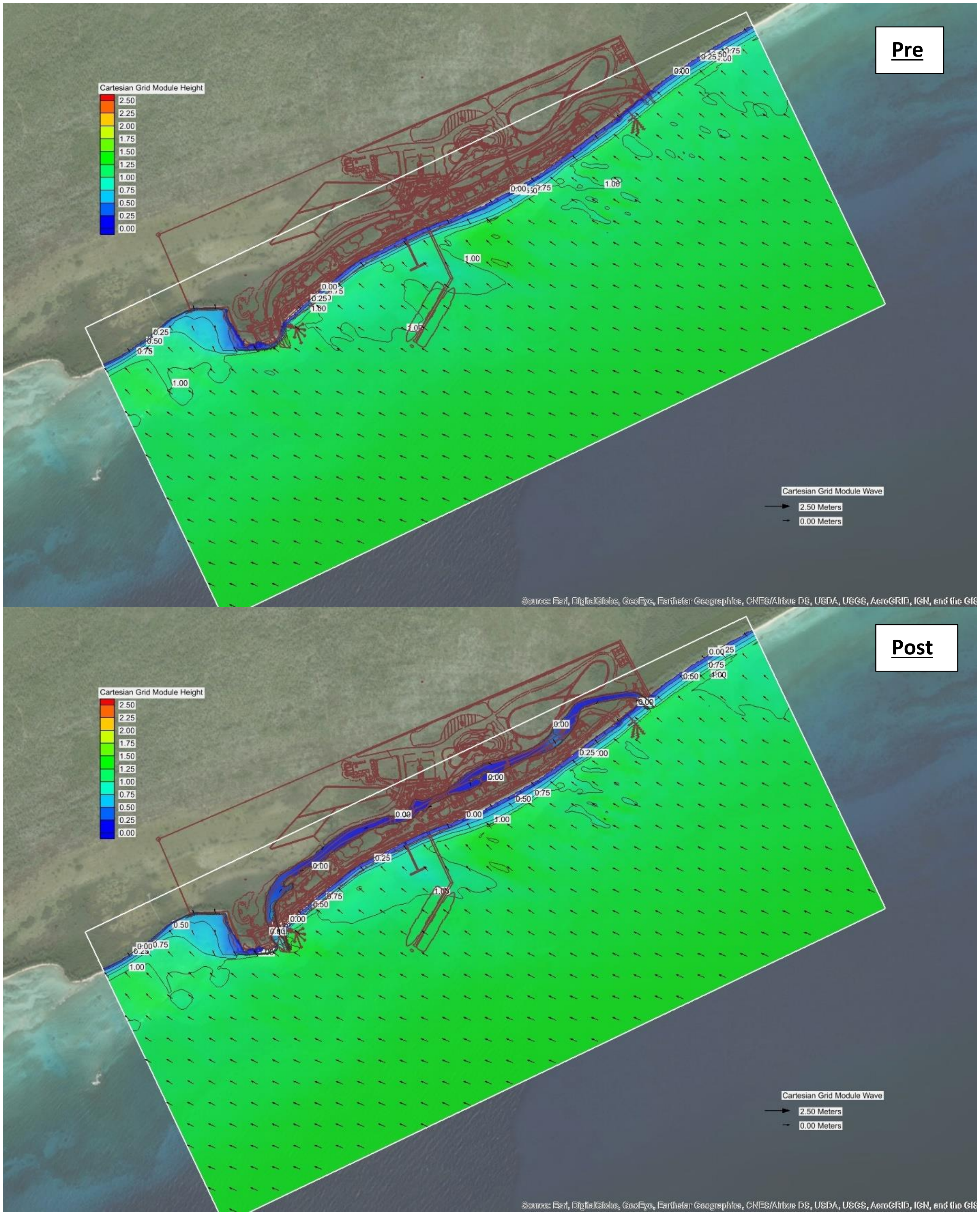


Figure 18. Case 3 (Typical ESE) Wave Height and Direction Outputs

Note Scale Change for Cases 4-9

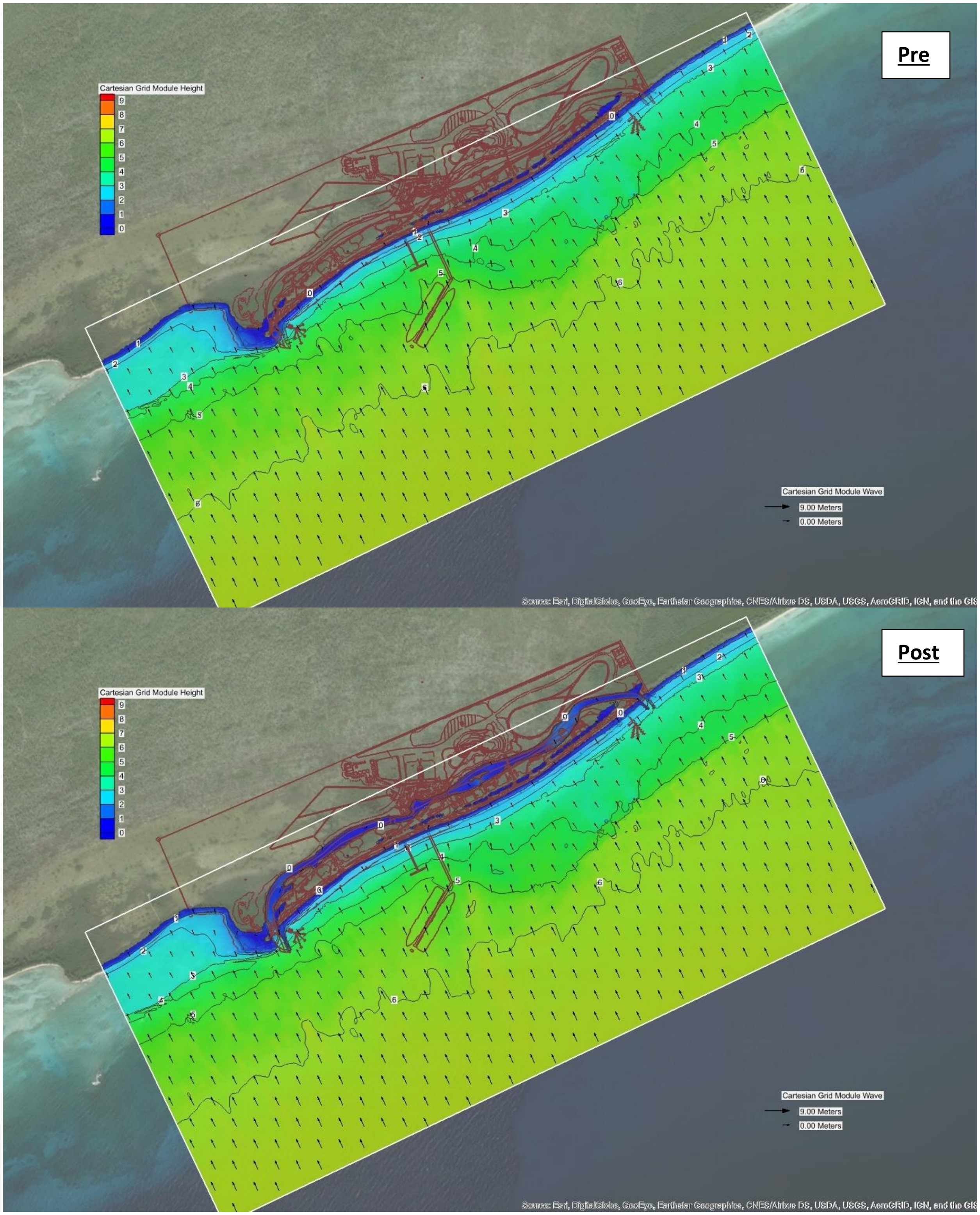


Figure 19. Case 4 (25-Yr SSE) Wave Height and Direction Outputs

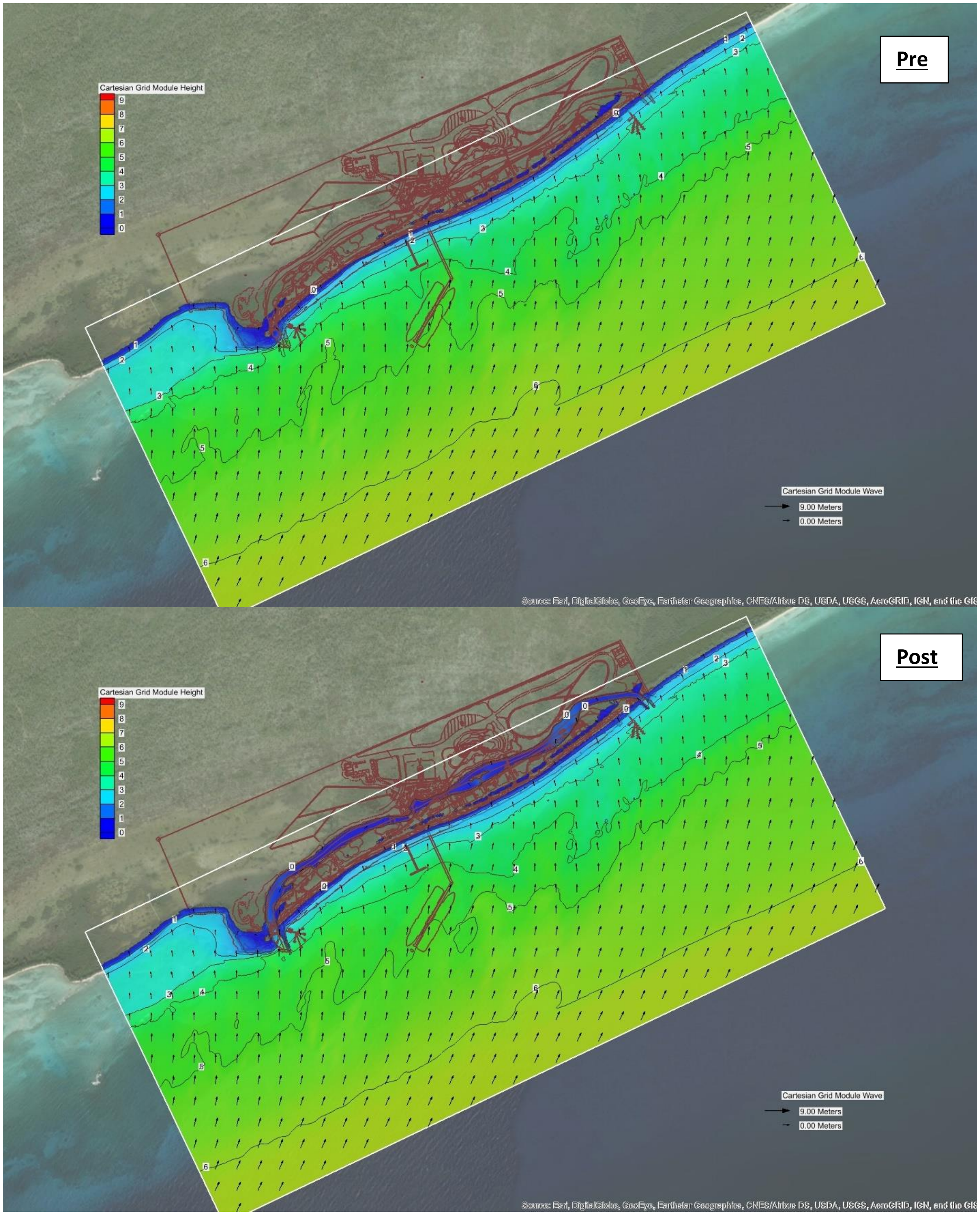


Figure 20. Case 5 (25-Yr SW) Wave Height and Direction Outputs

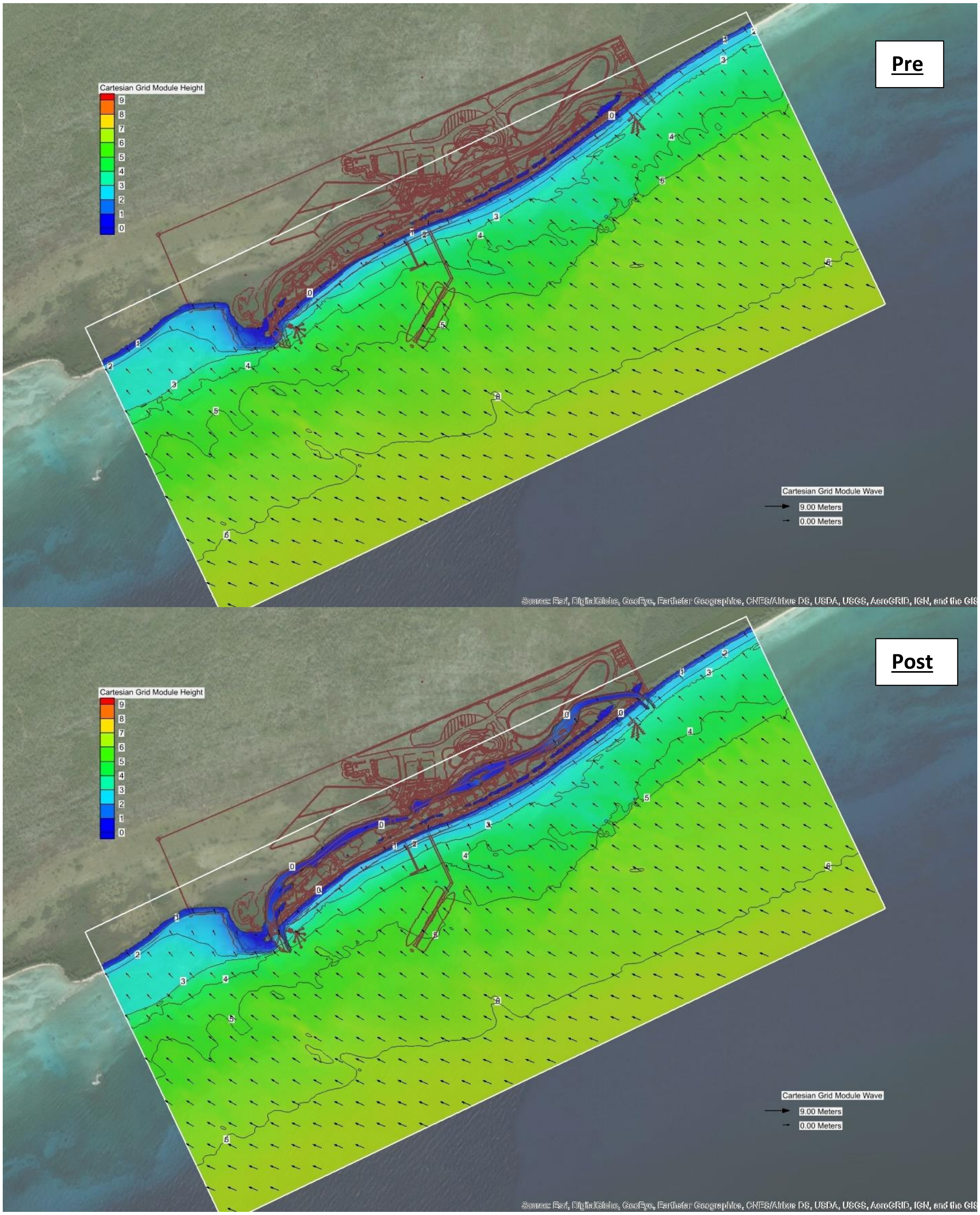


Figure 21. Case 6 (25-Yr ESE) Wave Height and Direction Outputs

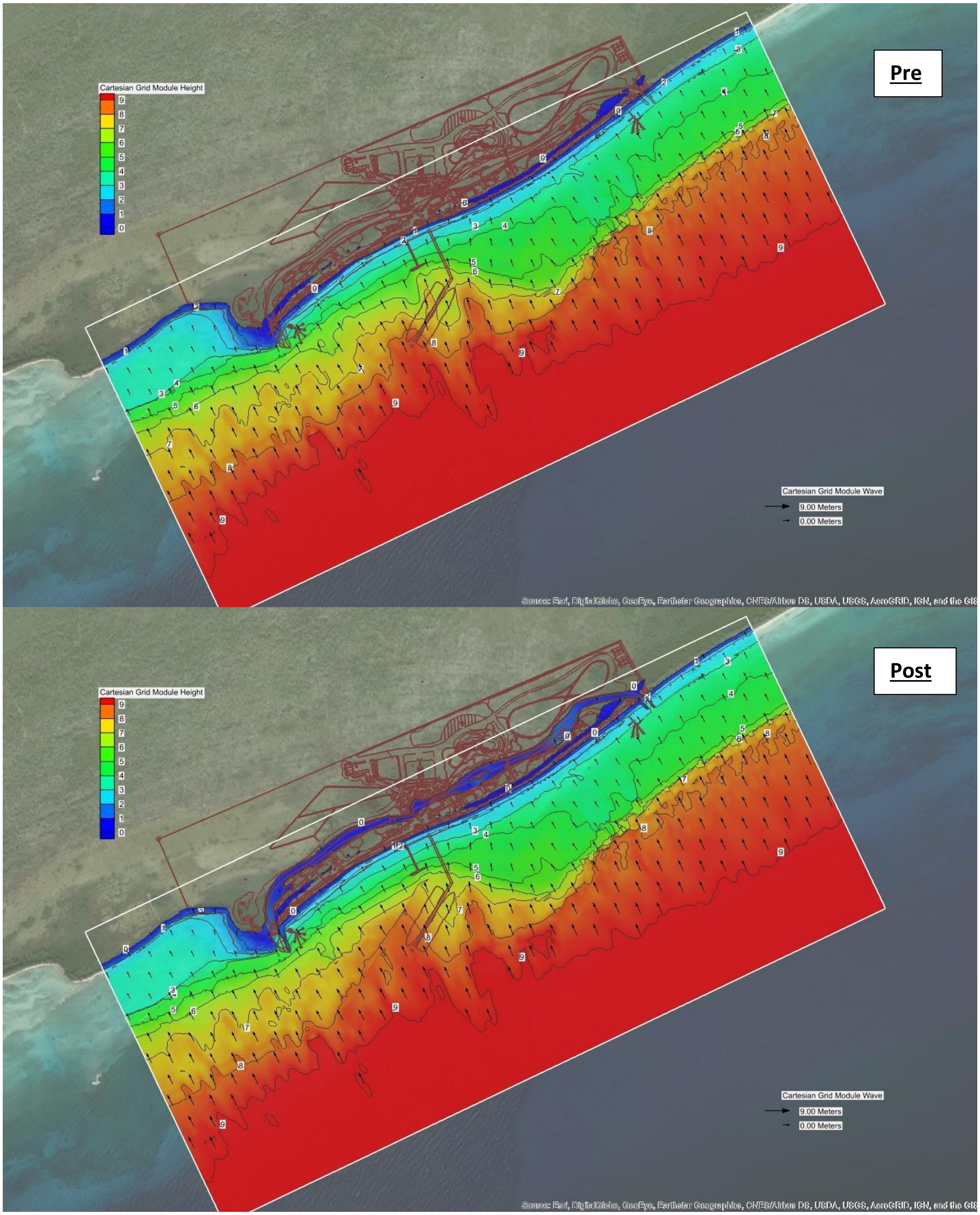


Figure 22. Case 7 (100-Yr SSE) Wave Height and Direction Outputs

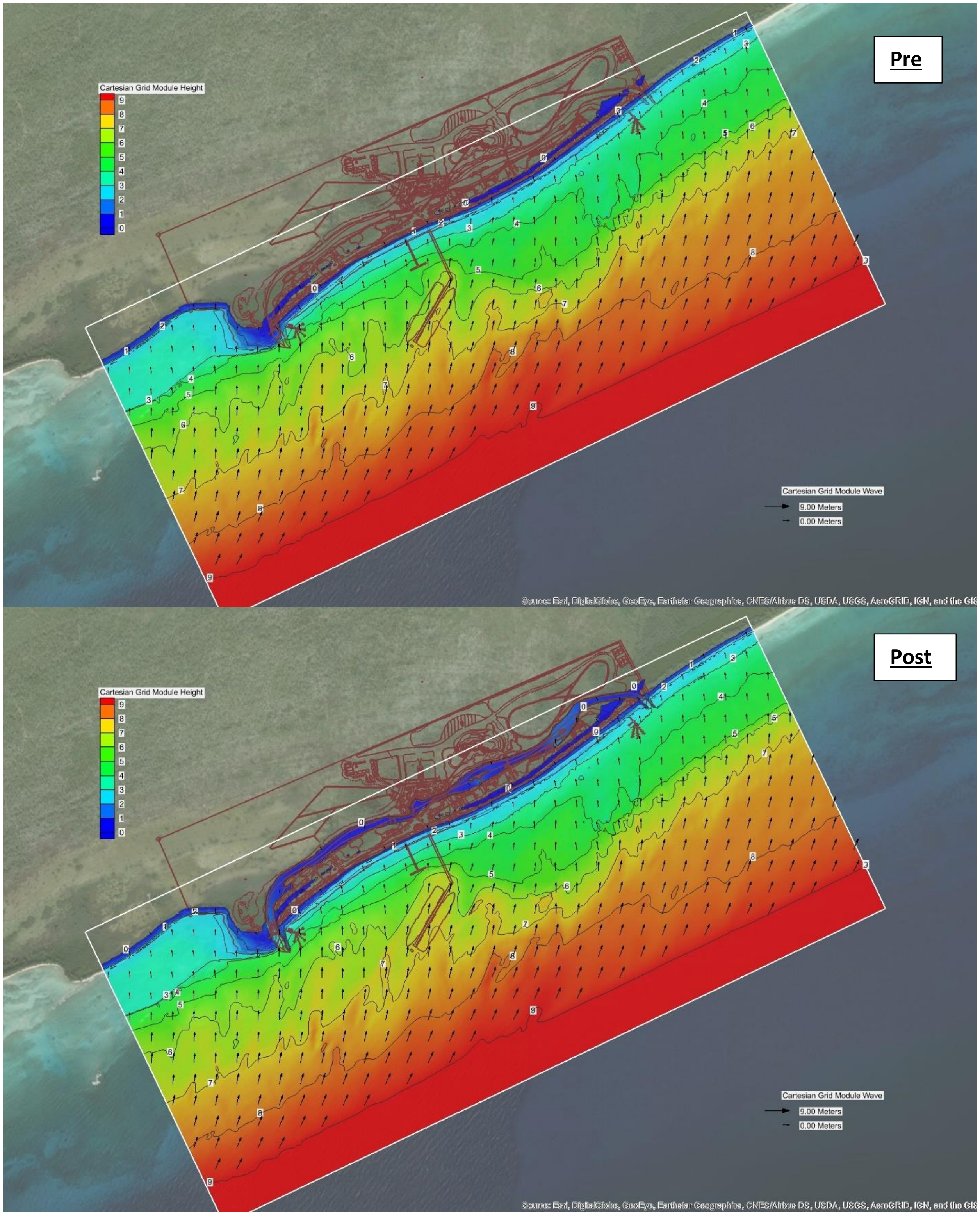


Figure 23. Case 8 (100-Yr SW) Wave Height and Direction Outputs

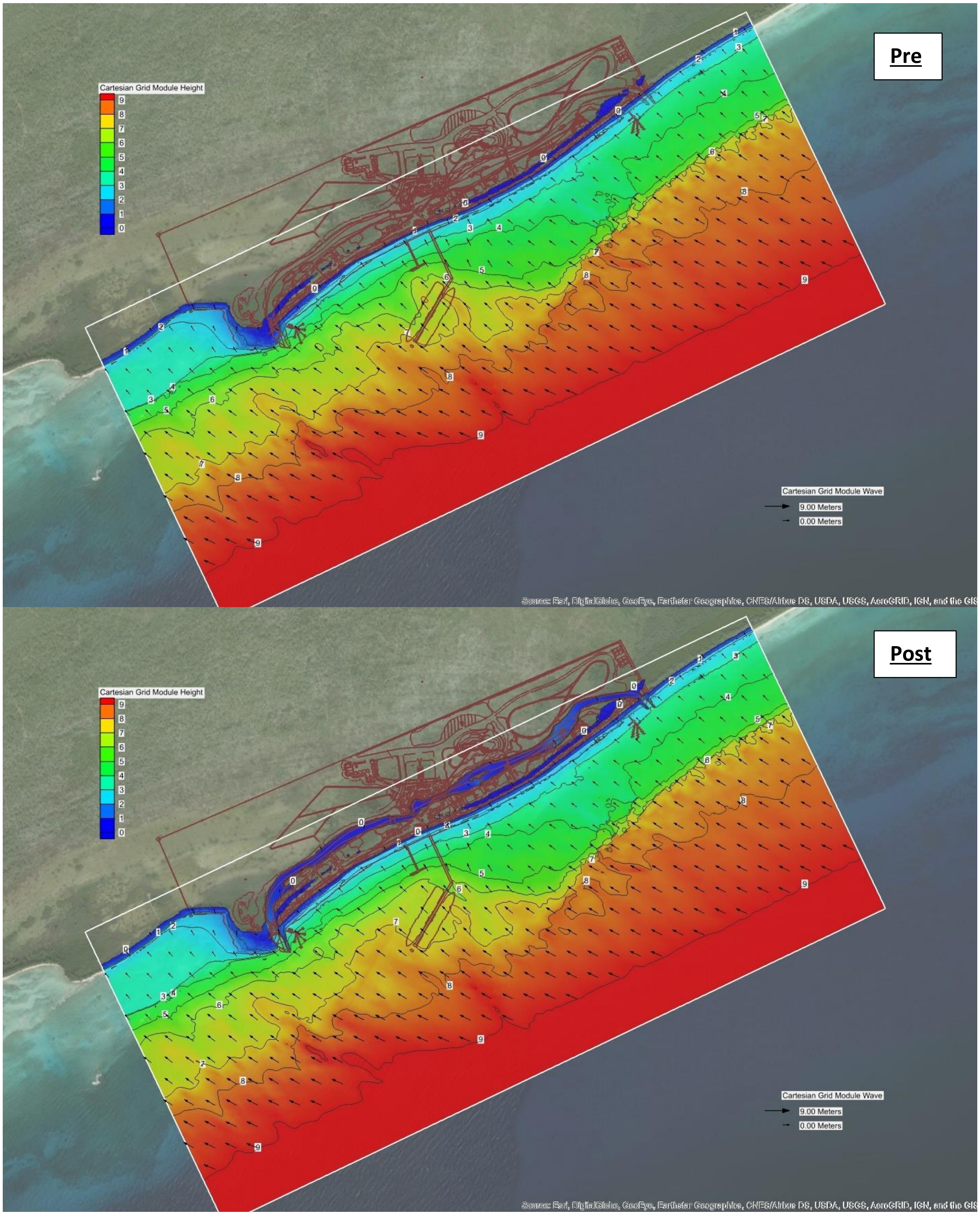


Figure 24. Case 9 (100-Yr ESE) Wave Height and Direction Outputs

Table 3. Pre-Project Model Output Station Wave Heights (meters)

Output Station	Case 1 Typical SSE	Case 2 Typical SW	Case 3 Typical ESE	Case 4 25-Yr SSE	Case 5 25-Yr SW	Case 6 25-Yr ESE	Case 7 100- Yr SSE	Case 8 100- Yr SW	Case 9 100- Yr ESE
1	1.12	1.00	1.05	5.80	4.86	5.16	8.57	6.58	7.31
2	1.09	0.95	1.03	5.39	4.76	4.92	7.19	6.39	6.51
3	1.16	0.95	1.04	3.20	3.07	3.17	3.44	3.44	3.44
4	1.12	0.96	0.95	3.21	3.21	3.21	3.45	3.45	3.45
5	1.05	0.85	0.87	4.27	3.67	3.99	4.83	4.40	4.83
6	1.02	0.88	0.88	4.52	4.21	3.99	5.62	5.45	4.97
7	1.02	0.82	1.02	2.47	2.40	2.47	2.71	2.71	2.71
8	1.13	1.05	1.00	2.93	3.00	2.76	3.19	3.26	2.99
9	1.14	0.90	1.00	3.17	3.05	3.07	3.47	3.37	3.35
10	1.12	0.88	1.04	3.08	2.90	3.07	3.37	3.29	3.36

Table 4. Post-Project Model Output Station Wave Heights (meters)

Output Station	Case 1 Typical SSE	Case 2 Typical SW	Case 3 Typical ESE	Case 4 25-Yr SSE	Case 5 25-Yr SW	Case 6 25-Yr ESE	Case 7 100- Yr SSE	Case 8 100- Yr SW	Case 9 100- Yr ESE
1	1.20	1.04	1.11	5.81	4.90	5.16	8.57	6.60	7.31
2	1.17	1.04	1.09	5.51	5.01	4.92	7.55	6.90	6.63
3	1.21	0.92	1.08	3.20	3.01	3.20	3.44	3.44	3.44
4	1.12	0.97	0.97	3.21	3.21	3.21	3.45	3.45	3.45
5	0.96	0.79	0.80	4.28	3.65	3.84	4.82	4.68	4.82
6	1.02	0.89	0.92	4.52	4.31	4.12	5.64	5.62	5.23
7	1.03	0.82	1.02	2.47	2.41	2.47	2.71	2.71	2.71
8	1.14	1.06	1.00	2.95	3.00	2.78	3.22	3.27	3.03
9	1.16	0.92	1.01	3.17	3.04	3.08	3.47	3.37	3.37
10	1.16	0.93	1.07	3.09	2.90	3.06	3.38	3.30	3.35