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CITY OF VIRGINIA BEACH

RUDEE WEIR STRUCTURE IMPROVEMENTS STUDY

Prepared for

City of Virginia Beach
2401 Courthouse Drive
Virginia Beach, VA 23456

Prepared by

Geosyntec Consultants, Inc.
2056 Vista Parkway, Suite 280
West Palm Beach, FL 33411

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Geosyntec consultants

Rudee Weir Structure Improvements Study

Basis of Design

Prepared for

City of Virginia Beach

Department of Public Works

Robert Neal, PE
Senior Engineer

Mike Jenkins, PhD
Senior Principal

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EXECUTIVE SUMMARY

The City of Virginia Beach requested a study to evaluate alternatives for improving sand bypassing operations in Rudee Inlet and increasing the resiliency of updrift Croatan Beach. The design alternatives considered additions and modifications to the existing weir system, including the following elements:

- The sheet pile weir;
- The sand trap;
- The south jetty;
- The north jetty
- The outer deposition area;
- The inlet channel, and
- The offshore breakwater.

The improvements must not increase sedimentation or the dredging frequency in the inlet channel or sand trap and may include structural elements. The total construction budget for the effort was projected to be $4 million. Increasing the resiliency on Croatan Beach targets approximately 1,000 feet (ft) of shoreline north of Twilight Lane and investigates methods to maintain a wider beach along this traditionally threatened oceanfront. The design also included consideration of performance improvements to the nourishment program along Croatan Beach from beneficial use of sand from dredging of the Atlantic Ocean Channel (AOC). **Figure ES-1** shows the weir system structures in planview.



Figure ES-1 – Rudee Inlet Weir Structures

The City assumed management responsibility of Rudee Inlet in 1989 with the dissolvement of the Virginia Beach Erosion Commission. In 2000, the City purchased a 14 inch hydraulic dredge (Rudee II) to bypass approximately 200,000 cubic yards per year (cy/yr) from the sand trap and inlet channel to Resort Beach. However, the maintenance need exceeds the historical production of Rudee II, and the City has purchased a more efficient dredge (Sandpiper) to begin operations in mid-2026. The process of purchasing and switching to a new dredge has allowed the City an opportunity to focus on the current operations and evaluate potential improvements.

Net northerly sediment transport quantities range from 250,000 cy/yr to 300,000 cy/yr, with the majority ending up in the Rudee Inlet sand trap and inlet system. The City receives help from the federal government through the United States Army Corps of Engineers (USACE) for maintaining safe navigation through the inlet. However, with the combined bypassing need reaching approximately 330,000 cy/yr, including southerly directed transport, the City continuously investigates new methods to help manage the inlet system.

The bypass material provides a beneficial use, assisting the City with nourishing the Resort Beach on the north side of Rudee Inlet. The City works year round to help remove material from Rudee Inet and help nourish the adjacent shoreline. The City maintains a hydraulic bypassing line to Resort Beach, but maintenance of Croatan Beach occurs from the USACE's periodic dredging of the AOC. The City has established a minimum beachfill template (MBT) for Croatan Beach to provide the necessary level of protection (LOP) to discourage direct storm impacts to upland infrastructure.

To assist the City with the management of Rudee Inlet and provide additional stabilization to Croatan Beach, the current study recommends the following improvements as shown in **Figure ES-2**.

- A 65 ft weir notch to focus transport into the sand trap;
- Dredge anchors along the sand trap updrift boundary to support efficient dredge operation;
- Restoration and 40 ft extension of the south jetty to limit sand losses from the sand trap into the navigation channel;
- Installation of a south discharge line to provide an alternate discharge location when discharge to the north beach is not practical or feasible; and,
- A 250 ft rock groin adjacent to the existing weir to retain more sand on Croatan beach and improve the performance of the beach nourishment program.



Figure ES-2 Rudee Inlet Weir Structures Proposed Improvements

ES-1 Bypassing Alternatives

The bypassing alternatives, as shown in **Figure ES-2**, promote sediment bypassing and increase the City's dredging efficiency. The 65 ft weir notch will focus sediment inflow into the sand trap Cuts 1 and 2, where the City's dredge operations prefer to work due to the wave sheltering provided by the offshore breakwater. The weir notch exists approximate to the rock groins seaward terminus and entails removing the existing sheet pile and cap from approximate elevation +1.8 ft referenced to mean lower low water (MLLW) [-0.7 ft referenced to the North American Vertical Datum of 1988 (NAVD)] to existing grade near -8 ft MLLW (-10.5 ft NAVD).

The proposed dredge anchors will also enhance bypassing operations by providing a quick setup for the dredge crew during repositioning movements. The dredge crew may connect to the installed dredge anchors as opposed to dragging a large anchor through the sand trap with a small work boat. The dredge anchors will be spaced on 65 ft centers and will mark both sides of the proposed weir notch. The City recently installed similar anchors through the inlet channel and along the north jetty. Thus, the proposed anchors will complete the coverage across the sand trap and inlet channel for the City's dredge operations.

Restoring and extending the south jetty will provide an estimated 4,000 cy of additional storage capacity in the sand trap. Increasing the sand trap holding capacity accomplishes multiple goals. The additional material provides a sediment pocket for dredge operations to target prior to moving into Cut 2 or Cut 3 and keeps the additional material from shoaling into the inlet channel.

The south discharge line would provide additional flexibility in operating hours for bypassing activities. Currently, bypassing to Resort Beach must be suspended during festivals, concerts, or other events. These interruptions can last a week or more, until Resort Beach becomes available again. Installing a south discharge line would allow bypassing to Croatan Beach and provide an alternative during such closures. It would also give the City the ability to address erosion hotspots along Croatan Beach when needed. Under this proposed alternative, two bypassing lines would be installed at locations identified by the City. The lines would be placed 6 ft below grade and landward of the +10 ft MLLW (+7.5 ft NAVD) contour to reduce the risk of storm damage and erosion impacts.

ES-2 Rock Groin

The rock groin serves as the primary alternative for increasing the Croatan Beach width and improving the performance of the beach nourishment program. Estimates show the 250 ft rock groin will provide an increased beach width through station 10+00 (Twilight Lane) as measured along the Croatan Beach baseline. The rock groin will anchor sediment on the updrift side to create a fillet that will continue to expand southward during normal weather patterns. The fillet should reach station 10+00 within a 5 year period and provide a 1 ft shoreline advance compared to the May 2024 shoreline position. The shoreline advance increases to an estimated 6 ft within a 10 year period.

The shoreline advance provides a more prominent benefit closer to the proposed groin, with an estimated 98 ft extension expected within 1 year of construction immediately adjacent to the groin. The shoreline advance reaches 26 ft within 3 years of construction at station 2+50 along the Croatan Beach baseline. **Table ES-1** and **Table ES-2** show the shoreline advance and corresponding volumetric gain expected from the 250 ft groin at the baseline stations shown in **Figure ES-3**.

Table ES-1: Rock Groin Estimated Shoreline and Volumetric Benefit (Baseline Stations -1+65 to 2+50)

Years Post Construction	Sta -1+65 (Groin) Shoreline Advance	Sta -1+65 (Groin) Volumetric Gain	Sta 0+00 Shoreline Advance	Sta 0+00 Volumetric Gain	Sta 2+50 Shoreline Advance	Sta 2+50 Volumetric Gain
1	98 ft	87 cy	24 ft	22 cy	1 ft	1 cy
2	137 ft	122 cy	71 ft	64 cy	15 ft	13 cy
3	137 ft	122 cy	82 ft	73 cy	26 ft	23 cy
5	137 ft	122 cy	94 ft	83 cy	42 ft	38 cy
10	137 ft	122 cy	106 ft	94 cy	65 ft	57 cy

Table ES-2: Rock Groin Estimated Shoreline and Volumetric Benefit (Baseline Stations 5+00 to 10+00)

Years Post Construction	Sta 5+00 Shoreline Advance	Sta 5+00 Volumetric Gain	Sta 7+50 Shoreline Advance	Sta 7+50 Volumetric Gain	Sta 10+00 (Twilight) Shoreline Advance	Sta 10+00 (Twilight) Volumetric Gain
1	0 ft	0 cy	0 ft	0 cy	0 ft	0 cy
2	1 ft	1 cy	0 ft	0 cy	0 ft	0 cy
3	5 ft	4 cy	1 ft	0 cy	0 ft	0 cy
5	14 ft	12 cy	3 ft	3 cy	1 ft	1 cy
10	34 ft	30 cy	15 ft	14 cy	6 ft	5 cy

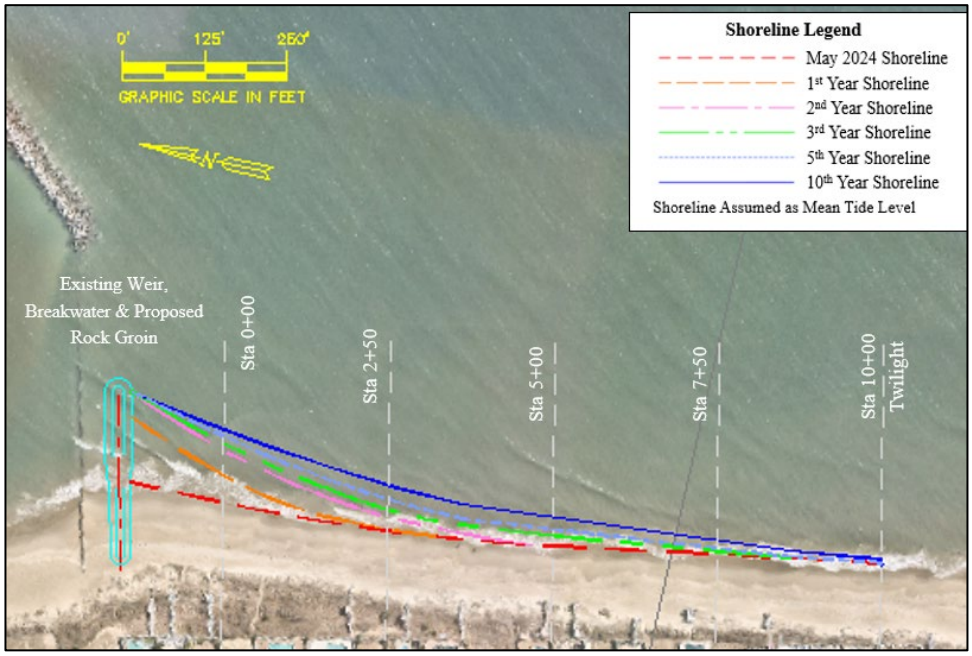


Figure ES-3: 10 Year Shoreline Advance Estimated for the 250 ft Rock Groin

The analysis shows the rock groin will also significantly reduce diffusion (end) losses associated with the placement of AOC maintenance spoil on Croatan Beach. The combined benefit provided by the reduced diffusion losses and the rock groin reach between 13 and 21 ft at Twilight Lane beach access over an anticipated 10 year period. The reduction in diffusion losses from AOC spoil placement will reduce the effective erosion rate from the beach fill template, improving overall project performance throughout the project area, including areas to the south of Twilight Lane. **Figure ES-4** provides a planview image of the shoreline and diffusion benefits expected for the rock groin and AOC nourishment.

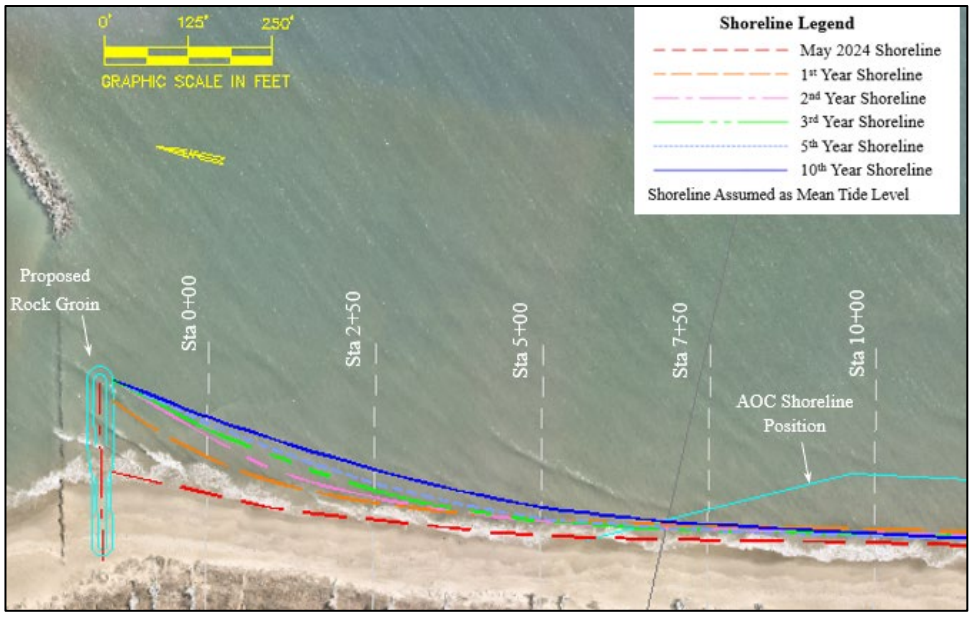


Figure ES-4: 10 Year Shoreline Advance Est. for the 250 ft Rock Groin and AOC (Sta -1+65 to 10+00)

ES-3 Stakeholder Considerations

The conceptual design was developed in consideration of input from multiple stakeholders. Key considerations included sediment transport around the breakwater, annual dredge volumes within the sand trap and inlet channel, aesthetic and construction-related effects, wave conditions influenced by the rock groin, navigation through Rudee Inlet, recreational beach safety, habitat impacts, and surfing. Additional considerations will continue to be addressed through the design process as the project advances.

To address considerations regarding sediment transport, the evaluation referenced numerical modeling completed during the September 2025 alternatives analysis. The work assessed potential changes in tidal current flow and magnitude relative to the documented net northerly transport rate of approximately 250,000 to 300,000 cy/yr. Results indicated no significant change under the proposed conditions; transport around the breakwater was estimated to increase by approximately 3%, which is minor relative to the approximately 17% annual variability historically observed. Consistent with this finding, annual dredging quantities in the sand trap and inlet channel should remain generally unchanged and may reduce long term, given anticipated performance improvements associated with beach nourishment to the south of the inlet. Bypassing efficiency is anticipated to improve though effectiveness in dredge operations. The design analysis uses best management practices and construction restrictions address remaining stakeholder considerations, including construction noise and visual effects, surfing and wave climate, navigation, beach safety, and habitat impacts. The analysis also recognizes construction monitoring should occur to confirm performance and support adaptive responses if additional considerations emerge.

TABLE OF CONTENTS

1. PURPOSE & NEED 1

2. CURRENT OPERATIONS 3
- 2.1 City Operated Dredge 3
- 2.2 USACE Government Plant Operations 4
- 2.3 Contract Dredging 5
- 2.4 Beach Nourishment Events 6
- 2.5 Croatan Beach Monitoring Program 6

3. PREVIOUS STUDIES 9
- 3.1 1983 Rudee Inlet Navigation Study (1983) 9
- 3.2 Beach Management & Restoration Plan (1988) 10
- 3.3 Senate Document 18; A Study of the Dredging at Rudee Inlet (1999) 10
- 3.4 Rudee Inlet Management Plan CIP 8-301 (2001) 11
- 3.5 Operational Needs of Dredging Operations Within the Rudee Inlet Basin (2004) 11
- 3.6 Rudee Inlet Dredging Efficiency Study (2006) 11
- 3.7 Wave Climate & Littoral Sediment Transport Study for Virginia Beach, VA – Rudee Inlet to Cape Henry (2008) 12
- 3.8 Rudee Inlet Weir & Groin Rehabilitation (2013) 13
- 3.9 Croatan Beach Shoreline Protection Assessment (2017) 13
- 3.10 Rudee Inlet Sediment Budget Analysis & Sand Trap Borrow Impacts Evaluation (2017) 14
- 3.11 Croatan Beach Nourishment; Coastal Modeling & Analysis Report (2018) 15
- 3.12 Beach Management Plan (Updated 2022) 16
- 3.13 Dredge Replacement Study (2022) 16
- 3.14 Croatan Beach Monitoring Reports (2017-2023) 16
- 3.15 Key Information 16

4. METOCEAN CONDITIONS 18
- 4.1 Wave Climate 18
- 4.2 Tides 20
- 4.3 Winds 20
- 4.4 Historical Storms 21
- 4.5 Extreme Storm Conditions 23
- 4.6 Currents 24

5. PREVIOUS ALTERNATIVE MODELING 28

6. RECOMMENDED ALTERNATIVE 31

6.1 Weir Notch 32
6.2 South Jetty Extension 33
6.3 Sand Trap Dredge Anchors 34
6.4 South Discharge Pipe 35
6.5 250-ft Total Length Rock Groin 35
6.5.1 Shoreline & Volumetric Benefit 36
6.5.2 Sea Level Rise 39

7. ADDITIONAL PROJECT CONSIDERATIONS 41
7.1 Sediment Transport Rates Around the Breakwater 41
7.2 Annual Dredge Volumes in the Sand Trap & Inlet Channel 42
7.3 Shoreline Evolution with the AOC 43
7.4 Aesthetic Landscape & Construction Activities 45
7.5 Wave Conditions Supported by the Rock Groin 46
7.6 Improved Navigation Access Through Rudee Inlet 47
7.7 Safety & Recreational Space on the Sandy Beach 47
7.8 Habitat Impacts 48

8. CONSTRUCTION SCHEDULE & COST 49

9. SUMMARY 51
9.1 Bypassing Improvements 53
9.2 Shoreline Enhancements 55
9.3 Additional Considerations 57
9.4 Estimated Cost 59

10. REFERENCES 60

LIST OF TABLES

Table 2-1: Croatan Beach Baseline Control 8
Table 3-1: Key Parameters Obtained from the Rudee Inlet Previous Studies 17
Table 4-1: NOAA Station 8639208 (Rudee Inlet) Tidal Datums 20
Table 4-2: Oceana Naval Air Station Wind Speed Annual % Occurrence (Iowa Mesonet) 21
Table 4-3: Severe Storms Within 60 Miles of Rudee Inlet Between 1980 and 2025 (NOAA 2026) 22
Table 4-4: FEMA Stillwater Elevation (SWEL) and Wave Conditions Approximate to Rudee Inlet (FEMA, 2015) 23
Table 4-5: Rudee Inlet CHS-SACS Best Estimate SWEL by Return Period 23
Table 6-1: Shoreline Evolution Analysis Constituents 37
Table 6-2: Rock Groin Estimated Shoreline and Volumetric Benefit (Baseline Stations -1+65 to 2+50) 37
Table 6-3: Rock Groin Estimated Shoreline and Volumetric Benefit (Baseline Stations 5+00 to 10+00) 38
Table 7-1: Combined Rock Groin and AOC Estimated Shoreline and Volumetric Benefit (Baseline Stations -1+65 – 2+50) 44
Table 7-2: Combined Rock Groin and AOC Estimated Shoreline and Volumetric Benefit (Baseline Stations 5+00 – 10+00) 45
Table 7-3: Combined Rock Groin and AOC Estimated Shoreline and Volumetric Benefit (Baseline Stations 12+50 – 20+00) 45
Table 8-1: Construction Conceptual Cost Estimate 49
Table 9-1: Groin Induced Shoreline Advance (Baseline Stations -1+65 to 10+00) 55
Table 9-2: Groin Induced Volumetric Gain (Baseline Stations -1+65 to 10+00) 56
Table 9-3: Combined Rock Groin and AOC Estimated Shoreline Benefit (Baseline Stations -1+65 to 10+00) 56
Table 9-4: Combined Rock Groin and AOC Estimated Volumetric Benefit (Baseline Stations -1+65 to 10+00) 56
Table 9-5: Stakeholder Considerations and Best Management Practice Responses 58

LIST OF FIGURES

Figure 1-1: Rudee Inlet Weir Structures and Features....1
Figure 1-2: Rudee Inlet Weir 2004 Profile View (Waterway, 2004) 2
Figure 2-1: Rudee Inlet Sand Trap Designations....4
Figure 2-2: Rudee II Dredge....4
Figure 2-3: Dredge Merritt Sidecast Dredging in Rudee Inlet 5
Figure 2-4: Croatan Beach Baseline and Monitoring Stations 7
Figure 3-1: Sediment Transport Along Virginia Beach (Briggs & Thompson, 2008) 13
Figure 4-1: WaveWatch III Wave Rose Analysis Offshore of Rudee Inlet 19
Figure 4-2: WaveWatch III Statistical Boxplot 19
Figure 4-3: Oceana Naval Air Station Annual Wind Rose (Iowa Mesonet) 21
Figure 4-4: Hurricane/Tropical Storms Within 60 Miles of Rudee Inlet Between 1980 and 2025 (NOAA, 2026) 22
Figure 4-5: ADCP Collection Transects 24
Figure 4-6: Measured Depth Averaged Mean Current Velocities (January 28, 2025)....25
Figure 4-7: Measured Depth Averaged Mean Current Velocities (January 29, 2025)....25
Figure 4-8: Measured Depth Averaged Edd Tide Current Velocities (Average Tidal Conditions) 26
Figure 4-9: Modeled Ebb Tide Current Velocities (Average Tidal Conditions)....26
Figure 4-10: Measured Depth Averaged Flood Tide Current Velocities (Average Tidal Conditions) 27
Figure 4-11: Modeled Flood Tide Current Velocities (Average Tidal Conditions) 27
Figure 5-1: Proposed Alternatives Plan View 29
Figure 5-2: Proposed Groin Alternatives Profile View 30
Figure 6-1: Proposed Weir Structure Improvements....31
Figure 6-2: Qualitative Example of Baseline Isolated Shoaling Patterns (Initial 8 Week Period)32
Figure 6-3: Qualitative Example of Isolated Shoaling with the Proposed Groin and Weir Notch33
Figure 6-4: Proposed South Jetty Improvements Planview....34
Figure 6-5: Proposed Pipe Pile Dredge Anchors (Plan View)....35
Figure 6-6: Analytical Analysis of Rock Groin 10 Year Shoreline Evolution....36
Figure 6-7: Typical Flood Tide Velocity Changes Estimated for the Proposed 250 ft Rock Groin 39
Figure 6-8: Rock Layers in a Typical Groin Cross-Section (Waterway, 2004) 40
Figure 7-1: Conceptual Design Percent (%) Change in Sediment Transport 42
Figure 7-2: Estimated Combined Shoreline Evolution (Proposed Groin and AOC)....44
Figure 7-3: Proposed Construction Staging and Access Area 46
Figure 7-4: Floating Pipeline Positioned in Rudee Inlet....47

Figure 9-1: Rudee Inlet Existing Weir and Breakwater.....51
Figure 9-2: Croatan Beach Area of Interest.....52
Figure 9-3: Proposed Bypassing Improvement Alternatives (Planview) 53
Figure 9-4: Shoreline Evolution with the Rock Groin and AOC (Sta 10+00 to -1+65).....57

LIST OF APPENDICES

Appendix A: September 2025 Numerical Modeling Assessment
Appendix B: Model Coefficients

ABBREVIATIONS

2D	two dimensional
ADCP	acoustic Doppler current profiler
AOC	Atlantic Ocean Channel
CIP	Capital Improvement Program
CHS	Coastal Hazard System
City	City of Virginia Beach, Department of Public Works
COLREG	International Regulations for Preventing Collisions at Sea, 1972
cy	cubic yard
cy/ft	cubic yard per foot
cy/yr	cubic yard per year
EA	environmental assessment
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
ft	feet
ft/s	feet per second
Hs	significant wave height
km	kilometer
LF	linear feet
LOP	level of protection
LS	Lump Sum
WRDA86	1986 Water Resource Development Act
m	meter
m/s	meters per second
M&N	Moffatt & Nichol
MBT	Minimum Beachfill Template
MHW	mean high water
MLW	mean low water
MLLW	mean lower low water
mph	miles per hour
MSL	mean sea level

NAVD	North American Vertical Datum established in 1988
NDBC	National Data Buoy Center
NOAA	National Oceanic and Atmospheric Administration
SACS	South Atlantic Coastal Study
SLR	sea level rise
Sta	Station
STWAVE	STeady-State Spectral WAVE model
SWEL	Stillwater Elevation
Tp	peak wave period
USACE	United States Army Corps of Engineers
Waterway	Waterway Surveying & Engineering, Ltd
WIS	Wave Information Study
WRDA	Water Resource Development Act
WW3	WaveWatch III

1. PURPOSE & NEED

The City of Virginia Beach (City) initiated a study to improve the management operations for the Rudee Inlet weir system and the updrift Croatan Beach. The study goals focus on increasing the beach width along approximately 1,000 feet (ft) of the updrift Croatan Beach without increasing the sedimentation rate and dredging frequency in the adjacent sand trap and inlet channel. The study evaluates alternate structural improvements to enhance the City's dredging operations for the inlet and channel system. Improvements to the dredging operations include channeling sediment flow into the sand trap in configurations to benefit the dredging operations and the mechanical bypassing rates for placing material on the Resort Beach. The study also considers multiple structural alignments incorporated into the existing weir system to anchor or capture sediment traveling from Croatan Beach. The captured material will extend the Croatan Beach width and provide additional stabilization adjacent to Rudee Inlet. The study evaluates approximately 1,000 ft of Croatan Beach, down to Twilight Beach Access. **Figure 1-1** shows the existing Rudee Inlet weir structures and key features of the inlet system.



Figure 1-1: Rudee Inlet Weir Structures and Features

The City improved the current weir system in 2004 by replacing the previous timber structure with a steel sheet pile wall ranging in vertical length from 25 ft to 35 ft. The weir maintains a concrete cap that transitions from elevation +10 ft referenced to mean lower low water (MLLW) [+7.5 ft referenced to the North American Vertical Datum established in 1988 (NAVD)] along the landward base to +1.0 ft MLLW (-1.5 ft NAVD) at the breakwater. The weir extends approximately 498 ft from the landward origination to the breakwater and slopes along the subaerial beach to elevation +8.8 ft MLLW (+6.3 ft NAVD) for approximately 53 ft. Along the foreshore slope of the beach, the weir cap slopes to elevation +1.8 ft MLLW (-0.7 ft NAVD) and maintains this elevation for approximately 145 ft in the nearshore zone. The remaining 189 ft, excluding the slope lengths, maintains the 1.0 ft MLLW (-1.5 ft NAVD) elevation through the surf

zone and into the breakwater junction. **Figure 1-2** shows a profile view of the existing weir system from the 2004 design drawings.

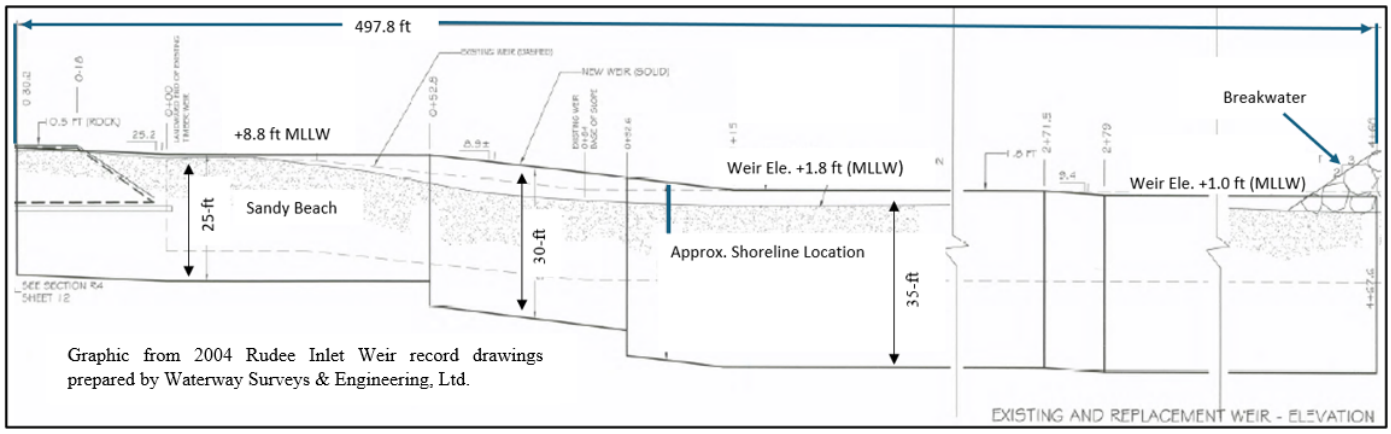


Figure 1-2: Rudee Inlet Weir 2004 Profile View (Waterway, 2004)

The weir's low profile across the nearshore and through the surf zone does not provide an adequate barrier to support a wider Croatan Beach. Transport escapes over the weir even within the sandy beach zone as wave uprush suspends the sediment particles and carries them into the downdrift sand trap. The weir appears to be performing as designed but modifications to the overall system may provide a cost effective approach for widening Croatan Beach.

2. CURRENT OPERATIONS

Rudee Inlet yields a diverse history of growth and transformation since its original construction in 1927 (Waterway et al., 2001). The inlet's original configuration focused on a small drainage canal connected to a tidal estuary. In the early 1950's, the Virginia Beach Erosion Commission dredged approximately 2 million cubic yards (cy) from the inlet to restore the Resort Beach immediately north of Rudee Inlet. From the original bypassing activities in the 1950's through present time, Rudee Inlet has served as a recreational and commercial resource worth maintaining for the local community. A federal study conducted in the late 1990's found the inlet contributed approximately $3.2 million annually to the local economy (Commonwealth of Virginia, 1999).

Sediment transport in the vicinity of Rudee Inlet predominantly carries south to north (Briggs & Thompson, 2008) and constantly threatens Rudee Inlet's stability. Estimates show the net south to north transport equals approximately 250,000 to 300,000 cubic yards per year (cy/yr) that must be managed by the inlet system (Waterway et al., 2001). The inlet weir system entails the actual weir, the offshore breakwater, and the south jetty. These features act together to capture transport in the sand trap and prevent the material from entering the inlet channel. The City also maintains additional programs to provide a regional sand management strategy for Rudee Inlet. The City operates a hydraulic dredge as needed to mechanically bypass material from the sand trap and inlet channel. The United States Army Corps of Engineers (USACE) supplements these efforts through routine maintenance with government plants. The USACE also contracts with private industry to maintenance the Rudee Inlet outer deposition basin. The bypassing operations generally place material on the Resort Beach but may also place material on Croatan Beach, pending the need. The City maintains an extensive beach nourishment and monitoring program to maintain a healthy shoreline on Croatan Beach. These efforts and initiatives provide several tools to assist the City with the long-term management of Rudee Inlet and the surrounding areas.

2.1 City Operated Dredge

The City gained maintenance authority of Rudee Inlet in 1989 with the dissolvement of the Virginia Beach Erosion Commission, which previously maintained two hydraulic dredges ranging from an original 8 inch discharge to 10 inch discharge by 1989. In 2000, the City purchased the Rudee II, which provides a 14 inch hydraulic discharge, to enhance maintenance activities that were inadequate to maintain Rudee Inlet. Estimates show the Rudee II bypasses approximately 200,000 cy annually to the Resort Beach [Moffatt & Nichol (M&N), 2017a] with a primary focus on maintaining the sand trap. The Rudee II operates inside the International Regulations for Preventing Collisions at Sea, 1972 (COLREG) lines, or landward of the breakwater, and within the sand trap feature.

The City has categorized the sand trap into three cuts to assist with identifying dredging and shoaling locations. The City references the sand trap areas as Cut 1 through Cut 3 as shown in **Figure 2-1**. Note, the cuts do not have a finite location but provide a general reference for the City to identify shoaling and dredging needs. The Rudee II does not maintain the operational stability to operate seaward of the breakwater and in most conditions must remain within the sand trap Cut 1 or Cut 2. The City purchased a new dredge (Sandpiper) in 2025 and anticipates bringing it online in mid-2026 (Tamayo, 2026). **Figure 2-2** shows the Rudee II conducting maintenance operations in summer 2025.

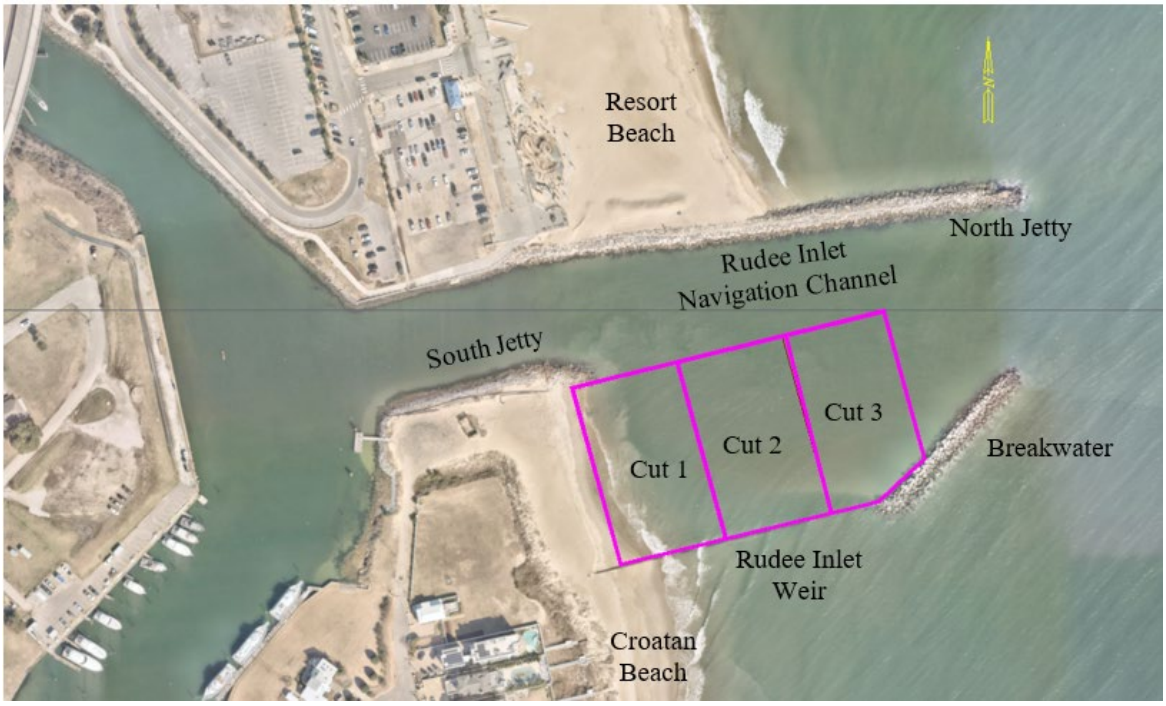


Figure 2-1: Rudee Inlet Sand Trap Designations



Figure 2-2: Rudee II Dredge

2.2 USACE Government Plant Operations

The USACE began partnering with the City to maintain Rudee Inlet in 1993 with the authorization of the federal channel. The inlet channel transitions from the -10 ft MLLW (-12.5 NAVD)

authorized depth in the entrance channel to -7 ft MLLW (-9.5 NAVD) depth in the inner channel. The channel width ranges from 110 ft at its maximum to a minimum of approximately 53 ft in the inner channel. The entrance channel extends approximately 1,065 ft and the inner channel extends 2,495 ft.

The USACE provides government plants to assist the City with maintenance operations several times a year, with recent estimates showing the USACE operations dredge approximately 28,400 cy/yr (M&N, 2017a). The USACE uses its fleet of shallow draft dredges including the sidecaster dredge Merritt as well as the split hull hopper dredge Murden. Historically, the USACE also used a second split hull hopper dredge, the Currituck. However, the Currituck's condition significantly restricts its current use and the USACE plans to replace the dredge entirely in the near future (Carter, 2026).

The hopper dredges completely remove material from the inlet complex, with nearshore placement occurring off Resort Beach in 12–20 ft of water. However, the sidecaster dredge Merritt only clears material from the immediate shoal and simply relocates the material approximately 100 ft outside the channel. Although the Merritt provides a vital service to maintain navigation in the inlet channel, the sidecast material may only be moved to the sand trap, which increases the need for the City to continue bypassing operations. **Figure 2-3** shows sidecast operations conducted in June 2023.



Figure 2-3: Dredge Merritt Sidecast Dredging in Rudee Inlet

2.3 Contract Dredging

The City and the USACE partner to contract with commercial companies to maintenance the outer deposition basin along with the sand trap and inlet channel. Contract dredging provides the largest benefit to the inlet system by providing large commercial dredges access to the sand trap and shoaling areas. Contract dredging also incurs the largest immediate cost because outside labor conducts the work. In efforts to maximize the benefit to the City's own staff and minimize long-term costs, the City and USACE schedule contract dredging as-needed when conditions stress the City's capabilities. Contract dredging operations remove approximately 100,000 cy from the outer

deposition basin and inlet channel during each maintenance event (M&N, 2017b). In 2022 a maintenance dredging effort removed approximately 190,000 cy from the Rudee Inlet navigation channel, sand trap, and outer deposition basin. The project placed approximately 50,000 cy on Croatan Beach (Waterway, 2024).

2.4 Beach Nourishment Events

The City benefits from beach nourishment opportunities in a similar manner compared to contract maintenance dredging for Rudee Inlet. Every 5–7 years, the USACE plans a nourishment event for the Resort Beach and generally combines the effort with the maintenance of the Atlantic Ocean Channel (AOC). In 2019, the Resort Beach received 1.4 million cy of sand from the AOC maintenance and in 2025 the project planned to place approximately 950,000 cy on the Resort Beach. However, the 2025 project placed only approximately 180,000 cy due to production issues involving finer material than anticipated. A portion of the remaining sand (approximately 130,000 cy) went on Croatan Beach, from Croatan Beach Park to just north of Twilight Lane (City, 2026).

2.5 Croatan Beach Monitoring Program

Due to the relative location of Rudee Inlet and the northward-directed longshore transport patterns, Croatan Beach is "updrift" of Rudee Inlet and therefore benefits from the weir-jetty system. As a result, Croatan Beach is relatively stable and historically experiences less erosion, although localized erosional hot spots occur. The City has regularly monitored Croatan Beach since June 2017 through pre-determined cross-shore profiles, on a biannual, and at times quarterly, basis. **Figure 2-3** presents an overview of the monitoring station baseline from Rudee Inlet to Camp Pendleton, and **Table 2-1** provides the state plane coordinates for the monitoring stations spaced on 500 ft intervals. Please note, multiple baselines exist for Croatan Beach; however, the City intends to transition to a single reference as described below.

The City monitors the beach and shoreline condition based on the recommended level of storm protection provided. A 2017 report estimated the level of protection (LOP) available based on the dune configuration and beach berm width. The study recommends a dune crest width equaling 15 ft at elevation +21 ft mean low water (MLW) (+23.4 NAVD) and a berm width of 50 ft. The report measures the berm width at elevation +8 ft MLW (+10.4 NAVD). (M&N, 2017a). Note, the City's monitoring program, conducted by Waterway Surveys & Engineering, identifies the berm height as +8.2 ft NAVD and identifies -16 ft NAVD as the seaward limit of major volumetric changes in the active profile (Waterway, 2024).

The City has established a Minimum Beachfill Template (MBT) based on the LOP results and actively pursues strategies to maintain the beach in excess of the MBT. **Table 2-1** provides the position and elevation of key points associated with the MBT and intends to compare future monitoring results as part of the periodic analysis (Tamayo, 2025). Note the key positions reference the seaward top of berm ranges or distances from the baseline along the profile azimuths and stations shown in **Figure 2-4**. The MBT maintains a 1:12 foreshore slope from elevation +5.75 ft NAVD to existing grade.

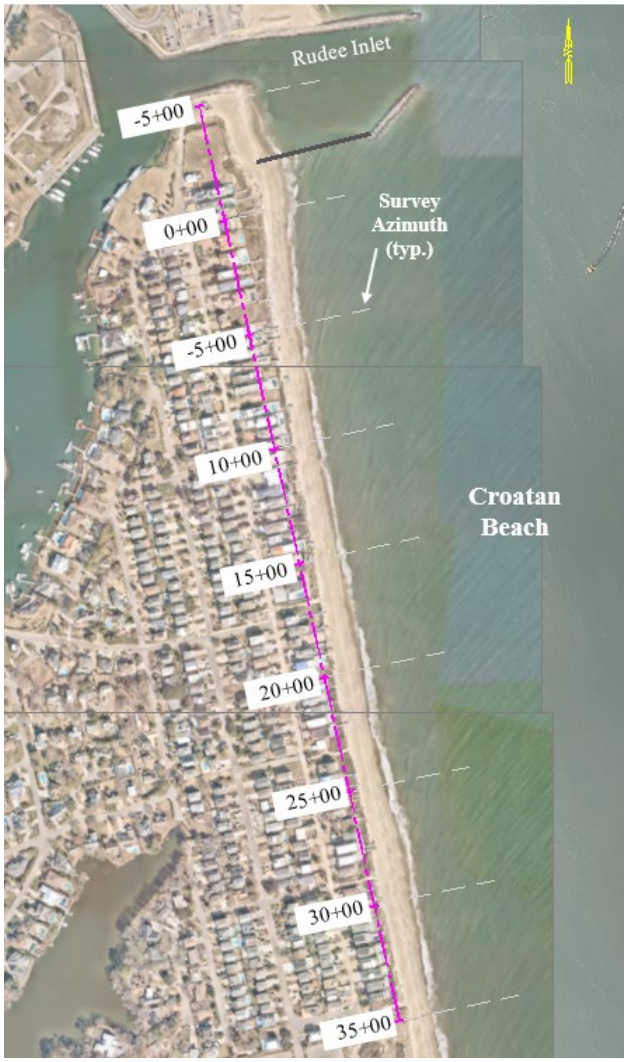


Figure 2-4: Croatan Beach Baseline and Monitoring Stations

Table 2-1: Croatan Beach Baseline Control

Station	Easting (ft)	Northing (ft)	Survey Azimuth (°)	MBT Top of Berm (ft)
-5+00	3,471,354.79	12,223,381.63	12	-
0+00	3,470,866.31	12,223,488.35	12	-
5+00	3,470,377.83	12,223,595.06	12	-
10+00	3,469,889.35	12,223,701.77	12	214.6
15+00	3,469,400.87	12,223,808.49	12	191.9
20+00	3,468,912.39	12,223,915.20	12	195.0
25+00	3,468,423.91	12,224,075.27	12	181.4
30+00	3,467,935.43	12,224,128.63	12	165.7
35+00	3,467,446.95	12,224,235.34	12	96.9

1. Azimuth measured counterclockwise from true east.
2. (-) indicates the MBT does not extend to the respective station.

3. PREVIOUS STUDIES

The City and the federal government have invested significant resources toward understanding and managing Rudee Inlet through several studies and reports dating many decades. The list below provides a sample of previous works conducted to better understand the coastal processes and stakeholder considerations surrounding Rudee Inlet. The items date to 1983 with a federal navigation study and include additional documents as follows:

- Rudee Inlet Navigation Study (USACE, 1983)
- Beach Management & Restoration Plan (Cubit Engineering, 1988)
- Senate Document No. 18; A Study of the Dredging of Rudee Inlet (Commonwealth of Virginia Senate Committee on Finance & House Committee on Appropriations, 1999)
- Rudee Inlet Management Plan CIP 8-301 (Waterway, Olsen Associates & Lent, L.K.,2001)
- Operational Needs of Dredging Operations Within the Rudee Inlet Basin (Waterway, 2004b)
- Rudee Inlet Dredging Efficiency Study (Waterway, 2006)
- Wave Climate & Littoral Sediment Transport Study for Virginia Beach, VA – Rudee Inlet to Cape Henry (Briggs & Thompson, 2008)
- Rudee Inlet Weir & Groin Rehabilitation (Olsen, 2013)
- Croatan Beach Shoreline Protection Assessment (M&N, 2017a)
- Rudee Inlet Sediment Budget Analysis & Sand Trap Borrow Impacts Evaluation (M&N, 2017b)
- Croatan Beach Nourishment; Coastal Modeling & Analysis Report (M&N, 2018)
- Beach Management Plan (Beaches & Waterways Advisory Commission, 2002 and updated in 2022)
- Dredge Replacement Study (M&N, 2022)
- Croatan Beach, Beach Monitoring Reports 2017 through 2023 (Waterway, 2024)

The following paragraphs provide the key findings and summaries of the documents provided.

3.1 1983 Rudee Inlet Navigation Study (1983)

The navigation study lays out the history of the management activities conducted to maintain Rudee Inlet and recommends a dredging plan to address issues associated with shoaling in the inlet and threats to navigation. The report includes a recommended channel template and environmental assessment (EA) of the proposed management activities to provide additional help maintaining safe navigation within the inlet channel. Main findings include the need for structured and consistent dredging in areas of the entrance channel and sand trap. The study reports an annual maintenance need of 220,000 cy/yr expected from the proposed improvements. The report

recommends bypassing maintenance material dredged from the entrance channel and sand trap to the north beaches.

3.2 Beach Management & Restoration Plan (1988)

The Beach Management & Restoration Plan recognizes beaches as a critical economic and environmental asset and provides a proactive approach for managing the City's shoreline. The recommended plan proposes to consolidate coastal management responsibilities into a single City department and suggests establishing a coastal planning committee to assist streamlining coordination among local, state, and federal agencies.

The plan evaluates Croatan Beach and the Resort Beach, along with five other shoreline segments separately based on their respective natural characteristics, erosion trends, and land use patterns. The plan recommends maintaining a 100 ft wide berm along Croatan Beach and anticipates nourishments on the order of 27,000 cy would be required on 3 year intervals. The shoreline near Camp Pendleton suffers from the most significant erosion rate and could lead to substantial losses under a Do-Nothing Alternative.

The report also discusses the potential of federal funding to offset the erosional losses. A federal review would be necessary to meet certain criteria. The project would most likely qualify for federal cost share under Section 933 of the 1986 Water Resource Development Act (WRDA86). For the Resort Beach, the study recommends integrating protective infrastructure (like seawalls) to maintain aesthetics, access, and visitor appeal of the ocean front property.

3.3 Senate Document 18; A Study of the Dredging at Rudee Inlet (1999)

This study reviews the development of Rudee Inlet from a small drainage feature to a major inlet supporting more than 148,000 annual vessel trips prior to the 2000's. Managing the inlet continues to be a complex engineering consideration balanced between the continuous sediment transport entering the inlet system and the navigation benefits retained by keeping the channel accessible. The study estimates the net northerly transport rate reaches over 200,000 cy annually from Croatan Beach.

Management of Rudee Inlet includes capturing or preventing the transport from closing the navigation channel while also encouraging bypassing sediment to maintain the Resort Beach. Maintenance dredging and mechanical bypassing has been ongoing since the 1950's through a combined effort from the City and federal government. Annual dredging costs to the City exceed a million dollars per year and do not fully address the inlet's stability during significant storms or high transport events.

The report concludes the maintenance responsibility should be shared between federal, state, and local stakeholders and notes the limited resources available from the state. Examples from states like Florida demonstrate alternative approaches, such as special taxing districts or dedicated funding streams for coastal management. The study underscores the need for improved long-term strategies such as structural betterments to reduce long-term maintenance costs while ensuring the safe and reliable operation of Rudee Inlet.

3.4 Rudee Inlet Management Plan CIP 8-301 (2001)

The Inlet Management Plan reviews the management history of Rudee Inlet, beginning in 1927, and focuses on a long-term management strategy to improve navigation, dredging efficiency, and economic performance. The inlet is a critical regional asset, providing the only navigable access to the Atlantic Ocean across a long stretch of coastline and supporting recreational boating, fishing, tourism, and waterfront development. However, persistent shoaling, hazardous navigation conditions, and aging infrastructure have limited its effectiveness and poses risks to users and local economic growth.

The plan estimates the average annual net northerly sand transport rate at about 250,000 to 300,000 cy/yr in the vicinity of Rudee Inlet, with north-directed transport approximately 3–8 times greater than south-directed transport. The transport drivers stem from the prevailing east-southeast wave approach and wave refraction created from the Chesapeake Bay shoals. Southerly transport values reach approximately 2,000 cy annually and stem from periodic winter storms that create high energy shoaling events.

The plan estimates the annual dredging needed to maintain Rudee Inlet is approximately 300,000 cy and provides alternatives to enhance management options. The alternatives include sand tightening the north jetty and installing a rock spur on the seaward end to discourage transport around the structure. The plan estimates the north jetty improvements should reduce the dredging needs by approximately 60,000 cy/yr. The plan recommends raising the landward portion of the timber weir present at the time and restoring the portions of the overall structure. Raising the weir would enhance the condition of Croatan Beach and improve the sand trap performance. The plan also recommends additional dredging strategies to improve efficiency and storage capacity in the sand trap. The proposed improvements should improve navigation safety and reliability while reducing the dredging need with minimal environmental impacts.

3.5 Operational Needs of Dredging Operations Within the Rudee Inlet Basin (2004)

The study evaluates the logistical and operational requirements necessary to sustain dredging activities within the Rudee Inlet system. The study focuses on the interaction between multiple dredging programs, including continuous City dredging, annual federal maintenance dredging, and periodic beach nourishment support operations. These activities require coordinated use of the inlet, channel, sand trap, and adjacent basins, with the City dredge Rudee II performing continuous sediment removal and bypassing, while contract and federal dredges operate seasonally or episodically. The inlet also serves as a critical staging and refuge area for offshore dredging projects, highlighting its dual function as both an operational work zone and protected harbor. The study concludes that maintaining adequate in-inlet staging and mooring capacity is essential for efficient dredging operations, and that additional restrictions may compromise both the economic viability and long-term maintainability of Rudee Inlet.

3.6 Rudee Inlet Dredging Efficiency Study (2006)

The study provides the history of Rudee Inlet from installation of a concrete flume in 1927 to the federal channel designation in 1986. Rudee Inlet received major transformations beginning in 1952, when a beach nourishment expanded the inlet and created Lake Rudee and Lake Wesley

while also enabling direct ocean access. During this period, jetties were constructed to stabilize the inlet, and over subsequent decades, numerous engineered improvements were added, including bypassing systems, jetty extensions, a weir and breakwater system, and a sand trap.

The report recommends prioritizing safe and reliable navigation, particularly by increasing focus on dredging the outer bar, which is the primary source of navigation hazards. A central recommendation is to maintain longshore sediment bypassing (approximately the natural transport rate) while allowing more flexibility for dredging operations, including modifying seasonal restrictions to permit dredging during favorable weather windows. Additional recommendations include improving data collection (e.g., GPS-based dredging records and wave monitoring) and enhancing coordination and communication among stakeholders. The study also emphasizes improved documentation of dredging activities, more accurate and consistent navigation advisories, and integration of structural and operational strategies to create a more efficient, adaptive sediment management program.

3.7 Wave Climate & Littoral Sediment Transport Study for Virginia Beach, VA – Rudee Inlet to Cape Henry (2008)

The 2008 study published by the USACE (Briggs & Thompson, 2008) serves as a basis for assessing sediment transport rates in the vicinity of Rudee Inlet for the current weir structure improvements study. The 2008 study uses a large-scale two-dimensional (2D) numerical model (i.e., STeady-State Spectral WAVE model, STWAVE) to provide an order of magnitude estimate of sediment transport rates calculated through empirical relationships. The model domain extends approximately 15 kilometers (km) along Virginia Beach, from just south of Rudee Inlet to the Chesapeake Bay entrance. Researchers analyzed wave parameters from a 10 year record to estimate the respective longshore littoral transport. **Figure 3-1** presents the results for the study area in terms of gross, north, south, and net transport and shows a net northerly transport rate ranging between 200,000–250,000 cy/yr at Rudee Inlet (Station 35).

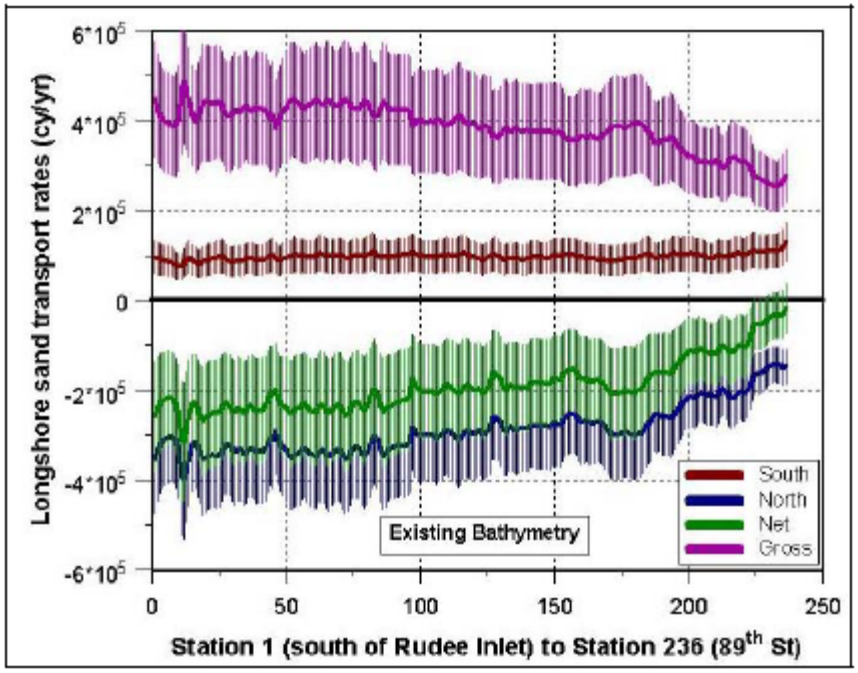


Figure 3-1: Sediment Transport Along Virginia Beach (Briggs & Thompson, 2008)

3.8 Rudee Inlet Weir & Groin Rehabilitation (2013)

The 2013 letter report provides recommendations for the Rudee Inlet weir and groin rehabilitation to address (a) the steel cap failure and potential structural instabilities at structures seaward end, (b) excessive erosion through the intertidal blow holes; (c) repair and relocation of the emergency access ramps; (d) re-use of existing misplaced stone; and (e) landward extension of the structure.

Observations noted the oversized steel cap along the weir suffered from wave uprush forces that have dislodged the cap. In addition, the intertidal blow holes appeared clogged and unable to relieve wave forces as designed. Bolt holes torched cut during installation also allows additional wave forces and instability in the cap. Observations of the dune erosion showed potential for improvements, however, the report does not note a specific design imperfection.

The report noted the replacement alternatives and methodologies for repairing the sheetpile cap and relocating the vehicle access ramp. The cap replacement provided significant details on the replacement alternatives and recommended a diagonally aligned rock access ramp. The preferred recommendations also include beach steps and vegetation as a future improvement. The report also provided other less favored recommendations to improve the observed conditions. The preferred alternatives range in cost at approximately $625, 000.

3.9 Croatan Beach Shoreline Protection Assessment (2017)

The 2017 shoreline protection assessment provides an LOP analysis based on the available material volume in the beach berm and dune. The LOP analysis estimates the maximum return interval storm sustainable by the input beach conditions. Results indicate the input conditions (2015) generally provide sufficient protection to habitable structures during storms up to the 100 year return period. However, weaker protection exists at specific northern locations near Twilight

Lane, where dunes are narrower and more vulnerable to erosion. The simulations also show that non-habitable structures in some areas could be impacted by storm surge and wave runup during more severe events, indicating localized deficiencies in the current beach profile.

The report further evaluates the effects of future sea level rise, specifically a projected increase of 1.5 ft over the next several decades. Under this scenario, model results show significantly greater dune erosion, increased overwash potential, and reduced levels of protection across much of the beach. In some areas, particularly near the northern end, even moderate storm events could lead to impacts on structures due to dune breaching and wave runup. These findings highlight the increasing vulnerability of Croatan Beach to coastal hazards if current conditions are maintained without intervention.

Based on these findings, the report recommends a conceptual beach nourishment and dune enhancement plan to improve both storm protection and recreational beach width. The proposed plan includes constructing a wider dune (approximately 15 ft wide at elevated height) and a beach berm of about 50 ft, requiring an estimated 130,000 cy of sand. Ongoing maintenance and periodic renourishment would be necessary to sustain the design profile, especially given observed erosion rates. The report concludes that coastal changes are primarily driven by the natural wave climate and sediment transport rather than dredging practices and emphasizes proactive management to maintain long-term shoreline resilience.

The assessment also evaluates historical dredge records to estimate the annual volume of material dredged from the inlet channel. The assessment utilizes data from 2006 to 2017 showing dredging events conducted by the City, USACE, and contract plants. The assessment estimates dredging removes between 200,000 and 330,000 cy/yr from the inlet system. The assessment reports that the City removes approximately 196,000 cy from the sand trap annually, and the USACE dredges approximately 28,000 cy/yr using government plants. Contract plants remove an average of approximately 42,000 cy/yr; however, contract plants operate on a 5–7 year frequency. The assessment also notes the City has permit authority to remove up to 350,000 cy/yr from the sand trap and 150,000 cy/yr from the outer deposition basin.

3.10 Rudee Inlet Sediment Budget Analysis & Sand Trap Borrow Impacts Evaluation (2017)

The report evaluates whether annually borrowing up to 20,000 cy of sand from the sand trap for dune repair at Croatan Beach would adversely affect federal navigation or the Virginia Beach Hurricane Protection Project. The study uses historical surveys, dredging records, and updated sediment-transport modeling to characterize how sand moves through the inlet system. As the document states, *"the annual rate of northerly-directed transport is typically more than three times the rate of southerly-directed longshore transport,"* establishing that Croatan Beach serves as the net sediment donor to the inlet and northern beaches.

Wave climate analysis relies on the National Data Buoy Center (NDBC) buoy 44099 and the USACE Wave Information Study (WIS) station 63199, covering 2008–2015 for measured data and 1980–2012 for hindcast data. Offshore waves predominantly approach from the east and east-southeast, with 94% of significant wave heights below 1.9 meters (m) and only 0.4% exceeding 3.3 m. Seasonal wave patterns occur in distinct manners: summer waves are smaller

and more southeasterly, while winter waves include stronger events from the east-northeast through the southeast. The winter wave patterns drive the longshore transport calculations used in the sediment budget.

Using the Family of Solutions method, the study generated thousands of valid sediment-budget scenarios and estimated the median gross littoral drift at approximately 475,000 cy/yr. The net northward drift approaches approximately 263,000 cy/yr. The study shows the historical dredging records align closely with this observation as the dredging requirement averages approximately 265,000 cy/yr. A large percentage of the shoaling originates from the south with an estimated 88–94% coming from Croatan Beach.

The study concludes that removing 20,000 cy/yr from the sand trap for Croatan Beach nourishment would have minimal system-wide impact. North of the inlet, the expected beach-profile change equals an approximate 6 inch profile lowering and 8–9 ft shoreline retreat over the 3,000 ft area of significant inlet influence. Effects diminish rapidly further north. The report estimated the southern back passing would not increase inlet shoaling or dredging frequency because the added sand on Croatan Beach would not significantly enhance transport over the weir. The study also estimates the impacts on the federal nourishment project would be negligible as the borrowed volume represents less than 8% of the annual inlet-derived supply and approximately 16% of the annualized external nourishment requirement, well within natural interannual variability.

3.11 Croatan Beach Nourishment; Coastal Modeling & Analysis Report (2018)

The coastal modeling analysis evaluates the Croatan Beach nourishment to estimate the expected shoreline evolution and maintenance interval necessary to maintain the design. The modeling shows higher end losses than typical due to the proximity of Rudee Inlet and relatively short project length. The weir structures help stabilize the shoreline north of Croatan Road but conflict with the inlet's shoreline influence. Diffusion losses to the south could also be lowered by extending the fill template into Camp Pendleton.

The analysis recommends a nourishment interval on the order of every 2–3 years to maintain the recommended design but also recognizes the additional cost to maintain such a program. The analysis cites the Rudee Inlet weir system as a main contributor to the inefficiencies of maintaining the design template. A greater initial beach width would be necessary to extend the nourishment interval. Sediment transport estimates for immediately post-construction conditions estimate the northerly quantities would increase approximately 80,000 cy/yr within approximately 500 ft of the weir system.

The analysis also estimates the sediment flow patterns over the weir and around the south jetty, showing an estimated 91.3% of the annual transport travels over the weir. Thus, approximately 8.7% travels around the breakwater. These values change to 90.2% and 9.8%, respectively, under the With Project conditions. The analysis also estimates the overall northerly transport would increase by approximately 26,000 cy/yr from the original estimate of 224,000 to 300,000 cy/yr, assuming natural bypassing factors remain lower than 15%.

3.12 Beach Management Plan (Updated 2022)

The beach management plan reviews the management practices for all the City's beaches and makes specific recommendations for improvements, including dedicated funding sources. The plan recommends immediate replacement of the timber weir in the original 2001 analysis; however, the update does not provide additional recommendations for the weir system. The original 2001 analysis notes the sand trap's authorized dredge elevation equals -20 ft MLW (-22.4 NAVD), and the annual bypass requirement from the inlet system ranges from 250,000 to 300,000 cy/yr based on the 2001 Rudee Inlet Management Study.

3.13 Dredge Replacement Study (2022)

The dredge replacement study evaluates replacing the Rudee II with a new cutter section dredge and estimates the benefits available by providing a customized design. The analysis shows Rudee II removes approximately 165,000 to 185,000 cy/yr from the inlet channel and sand trap; however, the need has increased to approximately 255,000 cy/yr. A main contributor for providing the increased production rate entails installing a walking spud for areas of low face (areas of lower sediment buildup above the authorized dredge depth). The report recommends replacing the Rudee II with an 18 inch customized cutter suction dredge to meet the City's needs.

3.14 Croatan Beach Monitoring Reports (2017-2023)

The monitoring reports discuss the annual volumetric and shoreline change measured along Croatan Beach, from Rudee Inlet to Camp Pendelton. The reports document storm events and nourishment activities that may influence the shoreline performance. Analysis of the shoreline trends stem from profile surveys taken along the project baseline at 250 ft intervals from the landward limit of the dry beach to an elevation of -16 ft NAVD to capture significant volumetric or position changes. The reports also define the mean high water (MHW) (+1.1 NAVD) and MLW (-2.2 NAVD) elevations for Croatan Beach. The City provided the survey files analyzed to determine the respective change rates with the written monitoring reports.

3.15 Key Information

The previous studies reviewed provide a strong background describing past management activities and the importance of Rudee Inlet to the City of Virginia Beach and national stakeholders. By providing the reports for review, the City has ensured the history and key data pointes may be carried forward into the current design efforts and potential future studies as Rudee Inlet continues to evolve and react to the coastal processes. **Table 3-1** provides a sample of the key information obtained from review of the previous studies and lists the relevant parameters used to advance the weir structures' improvements.

Table 3-1: Key Parameters Obtained from the Rudee Inlet Previous Studies

Parameter Name	Chosen Parameter	Relevant Source
Annual Net Sediment Transport at Rudee Inlet	250,000-300,000 cy/yr	Waterway Surveying & Engineering, Ltd. (2001); Briggs, M.K. & Thompson E.F. (2008); Moffatt & Nichol (2017)
Estimated Total Annual Dredge Volume from Rudee Inlet	200,000-330,000 cy/yr	2017 Croatan Beach Shoreline Protection Assessment; 1983 Rudee Inlet Navigation Study
Berm Elevation	+8.2 NAVD	Croatan Beach Monitoring Report, September 2023
Depth of Closure	-16 ft NAVD	Croatan Beach Monitoring Report, September 2023
Mean Low Water (MLW)	-2.2 ft NAVD	Croatan Beach Monitoring Report, September 2023
Mean High Water (MHW)	+1.1 ft NAVD	Croatan Beach Monitoring Report, September 2023

4. METOCEAN CONDITIONS

Recorded ocean conditions near the site provide a range of conditions to consider in the design effort. Conditions such as wave and wind predominant directions and intensities, along with surge elevations and historical storm data help illustrate the coastal processes influencing Rudee Inlet. The following paragraphs summarize the metocean data incorporated into this study as well as additional modeled data used to evaluate the proposed improvements. This analysis used a numerical model to evaluate potential changes in the coastal system. The modeling results provide critical results that must be interpreted correctly to assist decision makers with the management of the Rudee Inlet.

4.1 Wave Climate

The National Oceanic and Atmospheric Administration (NOAA) WaveWatch III (WW3) model helped characterize the wave climate in proximity to the site. WW3 provides a full-spectral model that outputs a time series of wave parameters such as the significant wave height (Hs), peak wave period (Tp), and wave direction to represent the average energy distribution of waves at a given location. The WW3 model has been thoroughly tested in the western North Atlantic, and the operational wave forecasting systems at NOAA rely on the WW3 model (Tolman, 2009). Data from WW3 covers multiple years, including 2005 to 2019. **Figure 4-1** presents a wave rose of the 2005 to 2019 dataset. The dataset shows waves approach Rudee Inlet primarily from the east and southeast directions, with wave heights typically less than 1 m (approximately 3 ft). **Figure 4-2** presents a boxplot from the 2005 to 2018 wave data (complete years only). The boxplot provides a graphical display that simultaneously describes several important features, such as center, spread, departure from symmetry, and identification of observations that lie unusually far from the bulk of the data (“outliers”).

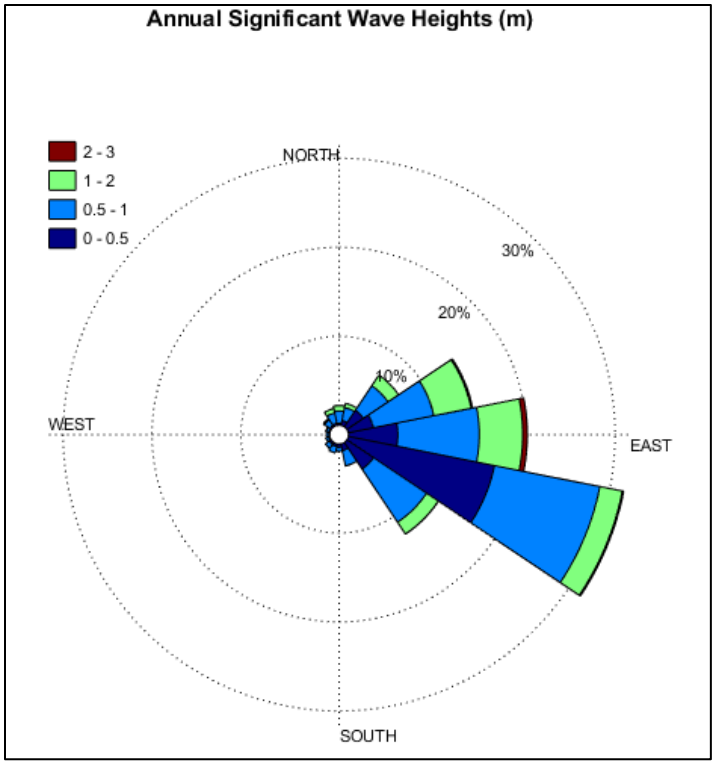


Figure 4-1: WaveWatch III Wave Rose Analysis Offshore of Rudee Inlet

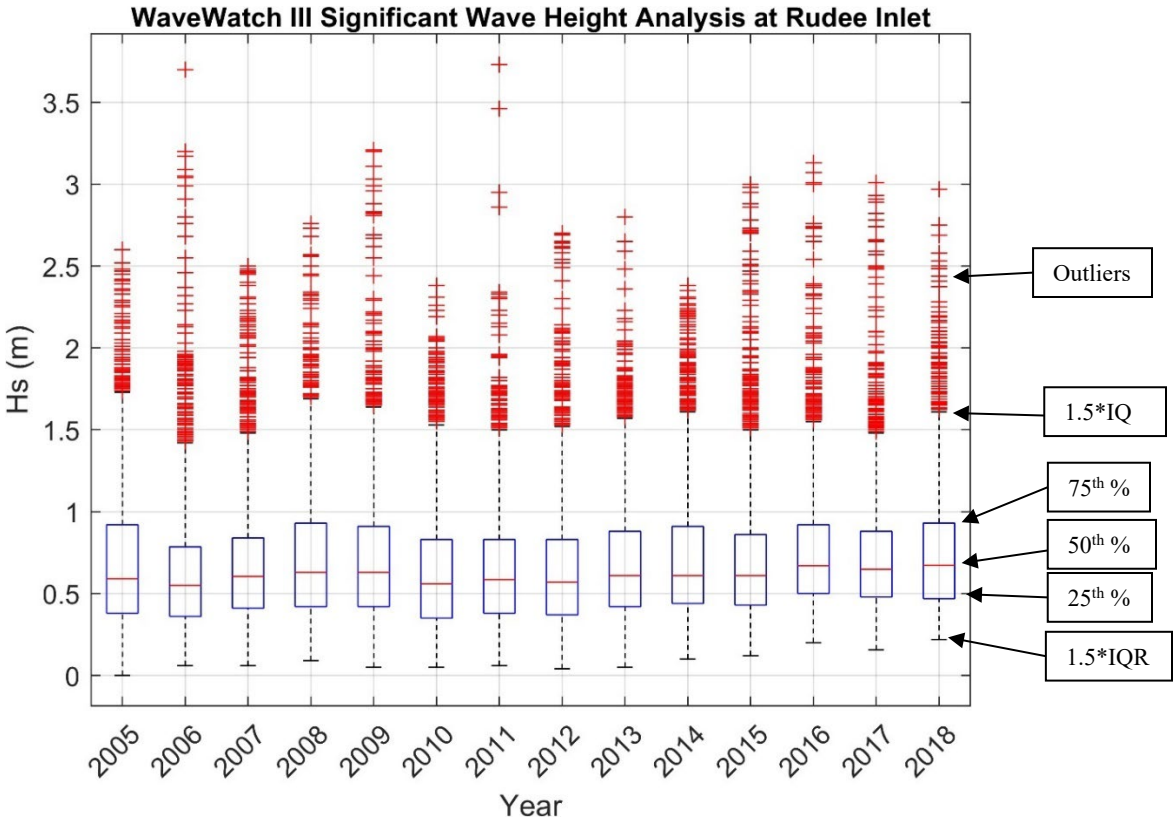


Figure 4-2: WaveWatch III Statistical Boxplot

4.2 Tides

Water levels at Rudee Inlet primarily stem from ocean tides under normal conditions. The nearest NOAA tidal datum station is located at Rudee Inlet Entrance (Station ID 8639208). **Table 4-1** shows the Rudee Inlet tidal datums in reference to NAVD, which approximates mean sea level (MSL). Tides in the area are semi-diurnal (two occurrences of high and low each day), with a mean range of approximately 1 m (3.3 ft) (MHW to MLW). Higher and lower water levels may occur due to regional wind/wave setup and direction. Other nearby NOAA sites with active gage deployments yield similar results.

Table 4-1: NOAA Station 8639208 (Rudee Inlet) Tidal Datums

Datum	Abbreviation	Water Elevation NAVD88, [ft (m)]
Mean Higher High Water	MHHW	1.21 (0.37)
Mean High Water	MHW	0.92 (0.28)
North American Vertical Datum of 1988	NAVD88	0.00
Mean Sea Level	MSL	-0.67(-0.21)
Mean Low Water	MLW	-2.36(-0.72)
Mean Lower Low Water	MLLW	-2.48(-0.76)

4.3 Winds

Typical wind conditions were evaluated using readily available wind data from the nearby Oceana Naval Air Station, which is approximately 3 miles west of the project site. Historical wind data was available at the air station from January 1970 through November 2024 (55 years). **Figure 4-3** shows a wind rose summary of this historical data, and **Table 4-2** provides a quantitative representation of the annual wind speeds by direction. Winds predominantly approach Rudee Inlet from the southwest and north-northwest direction. The higher, more predominant wind conditions from the southwest are likely driven by tropical storm systems during the late spring to summer months.

Measured wind conditions near the site present the predominant directions of short-fetch, wind-sea wave conditions, which is important for semi-closed basins such as Lake Rudee and Lake Wesley that are fetch-limited. Conversely, the main driver of wave conditions at Rudee Inlet and surrounding beaches is waves from offshore (swell), which are not dependent on local wind conditions.

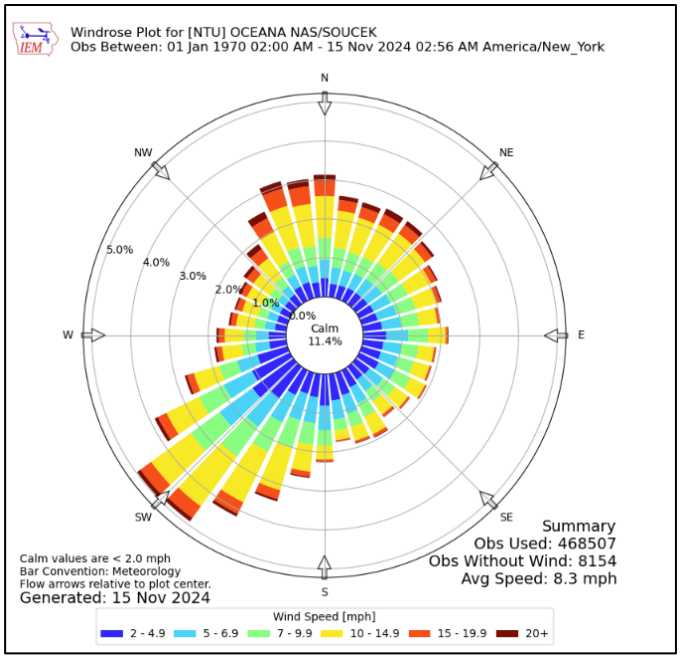


Figure 4-3: Oceana Naval Air Station Annual Wind Rose (Iowa Mesonet)

Table 4-2: Oceana Naval Air Station Wind Speed Annual % Occurrence (Iowa Mesonet)

Direction (Blowing from)	2.0 - 4.9 POOW in mph	5.0 - 6.9 POOW in mph	7.0 - 9.9 POOW in mph	10.0 - 14.9 POOW in mph	15.0 - 19.9 POOW in mph	20.0+ POOW in mph	Totals
North (335-24)	2.0	2.2	2.6	5.1	1.9	0.8	14.5
Northeast (25-64)	1.5	1.8	2.0	3.3	0.9	0.4	10.0
East (65-114)	2.6	2.5	2.3	2.3	0.3	0.1	10.1
Southeast (115-154)	1.9	1.7	1.5	1.7	0.2	0.1	7.2
South (155-204)	3.3	3.0	2.3	2.8	0.5	0.1	12.0
Southwest (205-244)	4.1	3.5	3.4	5.2	1.3	0.4	17.9
West (245-294)	2.3	1.8	1.7	2.6	0.8	0.3	9.4
Northwest (295 - 334)	1.3	1.1	1.2	2.4	1.1	0.5	7.5
Total	18.9	17.5	17.0	25.6	7.0	2.6	-

Notes:
POOW: percent occurrence of windspeeds
mph: miles per hour

4.4 Historical Storms

Hurricanes and tropical storms play a crucial role in the extreme wind climate at Rudee Inlet and can have acute impacts on beach erosion due to energetic wave conditions. The Atlantic Ocean hurricane season ranges from June through November each year. Since 1980 (45 years), a total of 15 tropical storms have passed within 60 miles of Rudee Inlet, 5 of which were a Category 1 hurricane or higher. That equates to a tropical storm passing the project location approximately

once every 3 years. **Figure 4-4** depicts the storm tracks of these 15 storms, and **Table 4-3** presents the name, year, and class of each storm. Though Hurricane Isabel did not pass within 60 miles of Rudee Inlet, it had a significant impact on Rudee Inlet in September 2003. Hurricane Isabel made landfall in the Outer Banks as a Category 2 hurricane but caused widespread damage reaching into Virginia. On Virginia Beach, Isabel significantly damaged the 15th Street fishing pier.

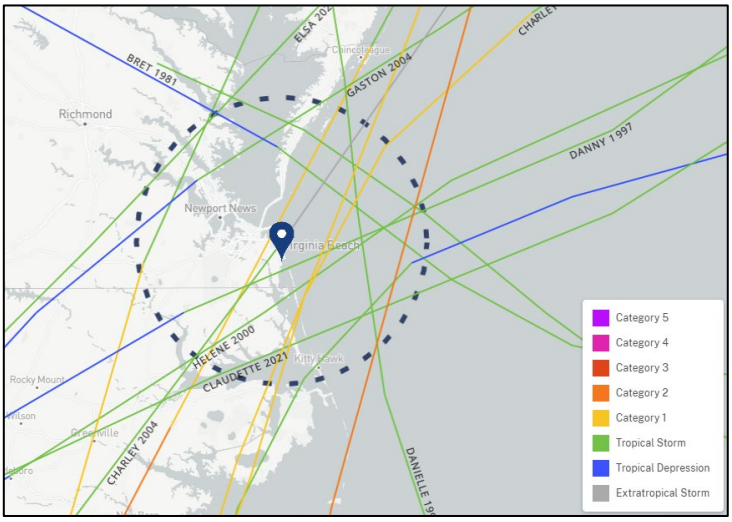


Figure 4-4: Hurricane/Tropical Storms Within 60 Miles of Rudee Inlet Between 1980 and 2025 (NOAA, 2026)

Table 4-3: Severe Storms Within 60 Miles of Rudee Inlet Between 1980 and 2025 (NOAA 2026)

Month, Year	Storm Name	Class
June 1981	Bret	Tropical Storm
September 1983	Dean	Tropical Storm
September 1985	Gloria	Hurricane, Cat 4
August 1986	Charley	Hurricane, Cat 1
September 1992	Danielle	Tropical Storm
July 1996	Bertha	Hurricane, Cat 3
July 1997	Danny	Hurricane, Cat 1
September 1999	Floyd	Hurricane, Cat 4
September 2000	Helene	Tropical Storm
August 2004	Charley	Hurricane, Cat 4
August 2004	Gaston	Hurricane, Cat 1
September 2007	Gabrielle	Tropical Storm
August 2011	Irene	Hurricane, Cat 3
June 2021	Claudette	Tropical Storm
June 2021	Elsa	Hurricane, Cat 1

4.5 Extreme Storm Conditions

Under extreme conditions, wave and water levels can reach much higher elevations than operational conditions. In the U.S., the Federal Emergency Management Agency (FEMA) provides extreme still water elevation (i.e., storm surge) and wave predictions through the Flood Insurance Study (FIS) program. At the site, the current effective FEMA FIS for the City of Virginia Beach, Virginia (dated January 16, 2015), provides an estimate for 10 year, 50 year, 100 year, and 500 year return period water levels and 100 year wave conditions (**Table 4-4**). It should be noted that FEMA flood maps are based on historical extreme events and do not account for sea level rise (SLR) or changes in storm patterns (intensity, frequency, etc.) due to climate change.

Table 4-4: FEMA Stillwater Elevation (SWEL) and Wave Conditions Approximate to Rudee Inlet (FEMA, 2015)

-	Starting 1% Annual Chance Wave Conditions	Starting 1% Annual Chance Wave Conditions	Starting Stillwater Elevation (ft NAVD)	Starting Stillwater Elevation (ft NAVD)	Starting Stillwater Elevation (ft NAVD)	Starting Stillwater Elevation (ft NAVD)
Transect No.	1% Significant Wave Height, Hs (ft)	1% Peak Wave Period, Tp (sec)	10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
40	13.2	13.5	5.1	6.5	7.1	8.3
41	13.7	13.4	5.2	6.6	7.1	8.4

Extreme wave and water level conditions can also be assessed through the USACE Coastal Hazard System (CHS). The CHS provides a platform designed to support coastal risk assessments through standardized hazard data and probabilistic analyses. The CHS integrates historical observations, advanced modeling, and statistical methods to estimate storm surge and wave conditions across U.S. coastlines.

As part of the South Atlantic Coastal Study (SACS), CHS statistical outputs for still water level elevations (SWELs) were extracted at the study site at model node ID 16961 (at Rudee Inlet). The SWEL presented here represents storm surge and wave setup but does not include the astronomical tide. At high tide, we can assume the SWEL would be higher by +0.92 ft. **Table 4-5** presents the statistical results of the respective SWELs and significant wave height by return period.

Table 4-5: Rudee Inlet CHS-SACS Best Estimate SWEL by Return Period

Return Period (years)	SWEL (ft NAVD), No Tide	Significant Wave Height (ft NAVD)
10	4.57	9.27
50	6.80	10.06
100	7.92	10.35

4.6 Currents

A current prediction station is located approximately 4.7 miles north of Rudee Inlet and 0.5 miles offshore, the Cape Henry Light Station (ACT4501). For the year 2025, the maximum predicted ebb and flood tide speed equals -1.6 knots and 2.3 knots, respectively.

In addition to nearby NOAA/state current gage stations, current and flow data assisted with the numerical model calibration referenced above. A boat mounted acoustic Doppler current profiler (ADCP) collected two days of data on January 28-29, 2025, capturing both ebb and flood tides under average tide range conditions. **Figure 4-5** shows the locations of sampling during the field survey. **Figure 4-6** and **Figure 4-7** present a graphical representation of the average current speed collected across each transect at each collection time (representing multiple collection intervals per transect).



Figure 4-5: ADCP Collection Transects

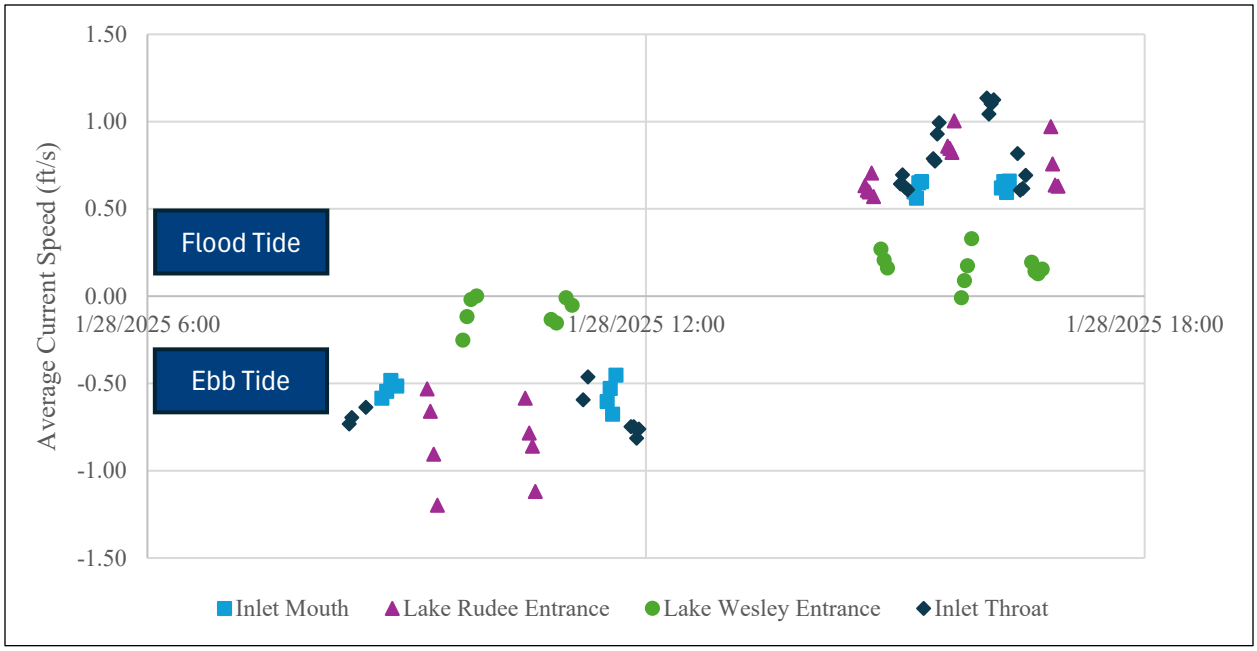


Figure 4-6: Measured Depth Averaged Mean Current Velocities (January 28, 2025)

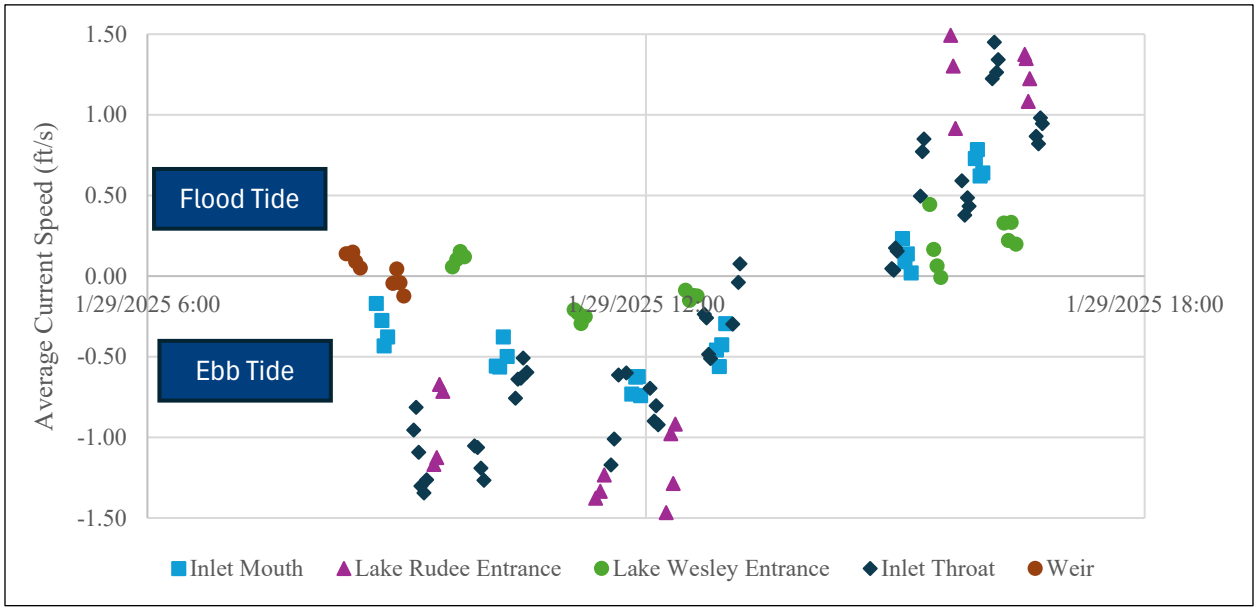


Figure 4-7: Measured Depth Averaged Mean Current Velocities (January 29, 2025)

The Rudee Inlet model calibration compared measured and modeled water elevations and current speeds over a 1 month simulation period. Current speeds, or tidal velocities, showed an acceptable comparison between modeled and measured records. **Figure 4-8** shows tidal velocities collected along cross-sections within the project area for ebb, or outgoing tide conditions. Vectors shown are depth-averaged (noting that surface currents can be much stronger), with the vector length and color scaled by magnitude. **Figure 4-9** shows a simulated comparison of ebb velocities across the model domain. **Figure 4-10** and **Figure 4-11** show a similar comparison for flood, or incoming tide conditions. Overall, currents are moderate, with peak ebb and flood tides about 1.5 feet per second (ft/s).

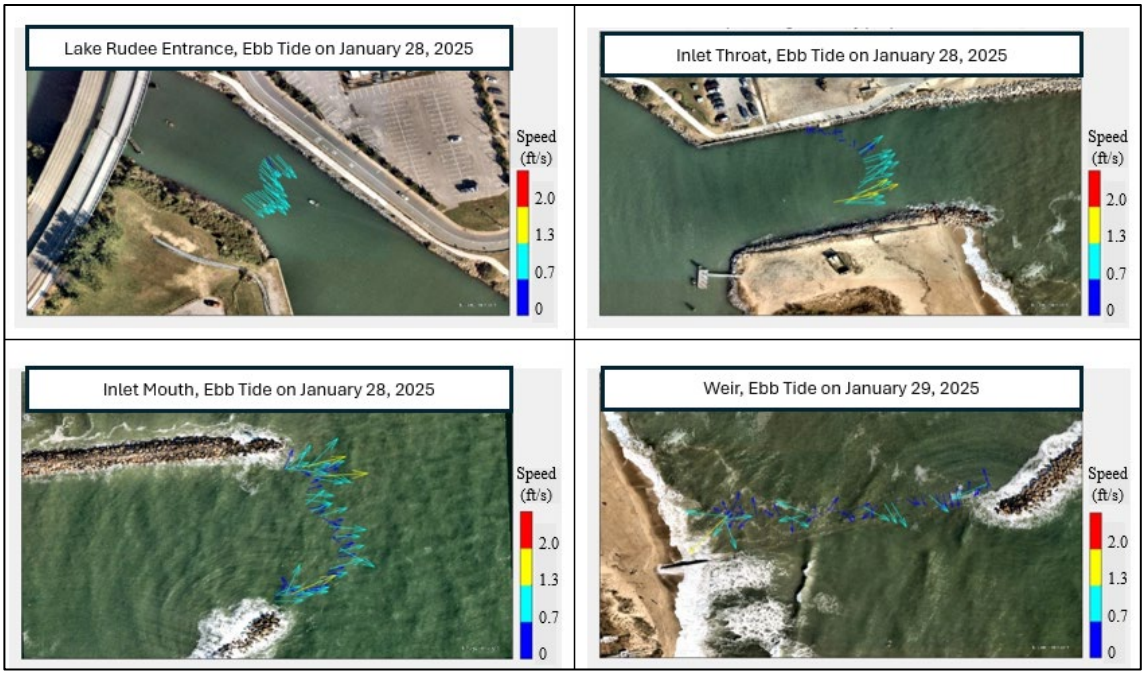


Figure 4-8: Measured Depth Averaged Edd Tide Current Velocities (Average Tidal Conditions)

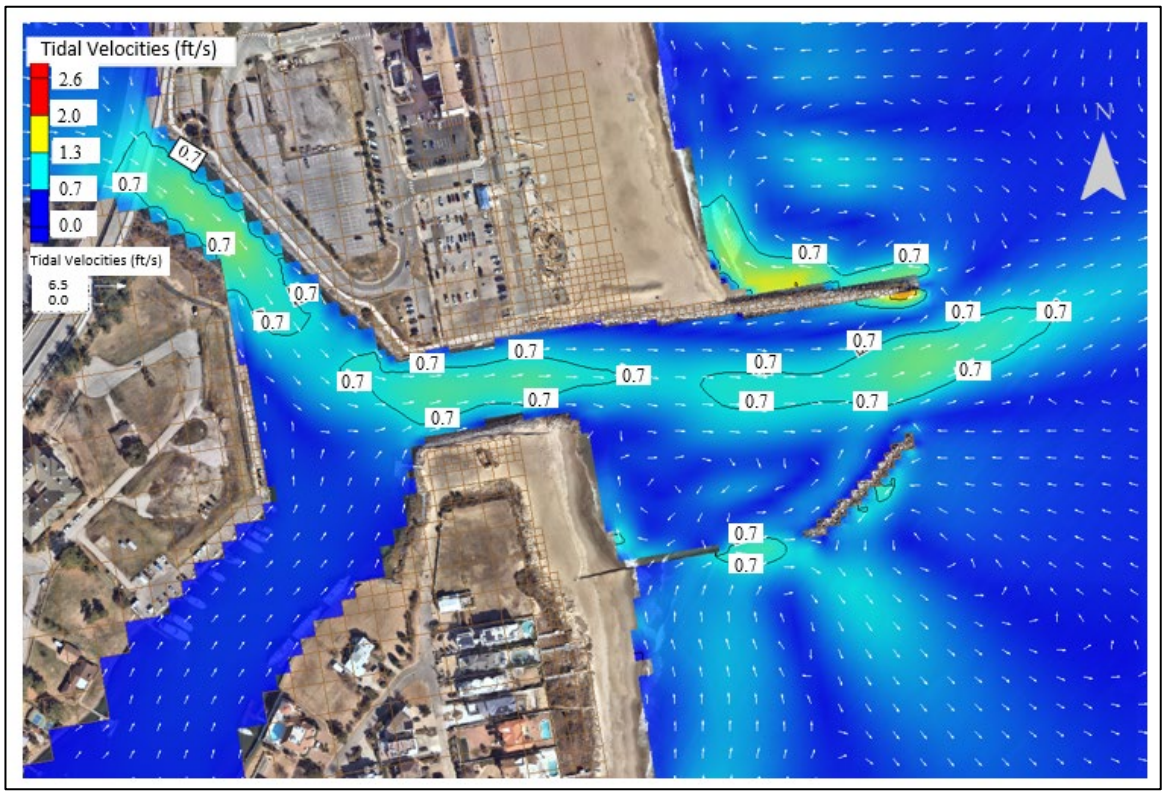


Figure 4-9: Modeled Ebb Tide Current Velocities (Average Tidal Conditions)

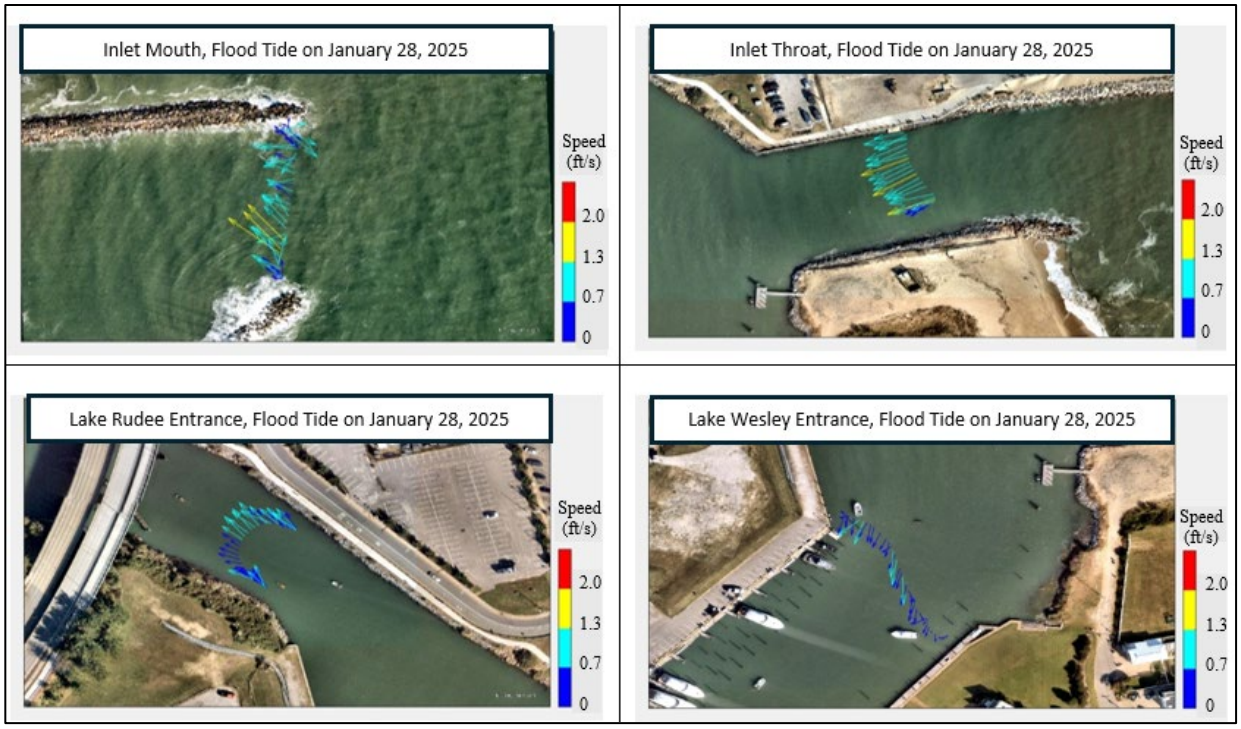


Figure 4-10: Measured Depth Averaged Flood Tide Current Velocities (Average Tidal Conditions)

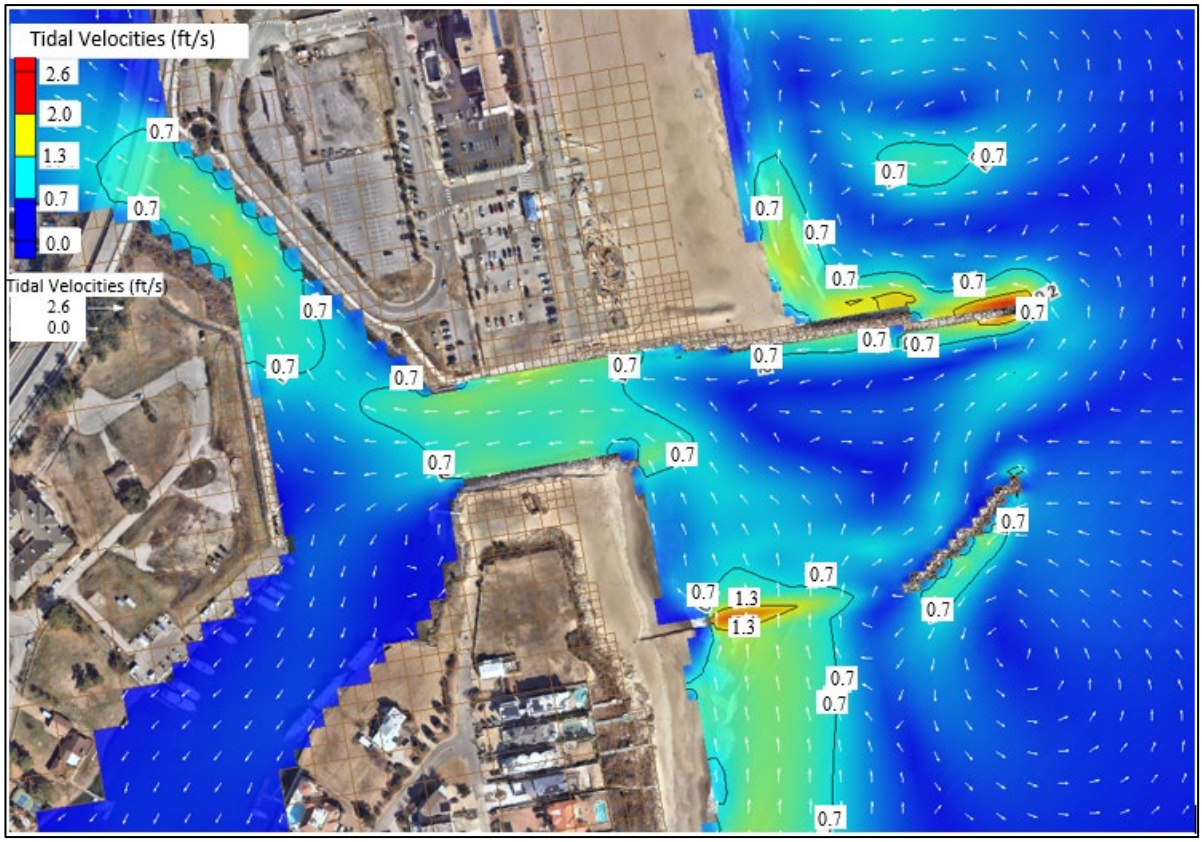


Figure 4-11: Modeled Flood Tide Current Velocities (Average Tidal Conditions)

5. PREVIOUS ALTERNATIVE MODELING

The previous alternative analysis conducted in September 2025 evaluated a total of nine alternatives or cases to compare how each might change the City's bypassing operations and the potential shoreline benefits for Croatan Beach. The list below summarizes the nine cases evaluated and **Appendix A** provides the numerical modeling assessment from the September 2025 analysis.

- **Case 1:** 2 0m (~65 ft) effective groin length extension only (90 m/~295 ft length weir remaining)
- **Case 2:** 30 m (~100 ft) effective groin length extension only (80 m/~260 ft length weir remaining)
- **Case 3:** 40 m (~130 ft) effective groin length extension only (70 m/~230 ft length weir remaining)
- **Case 4:** 50 m (~165 ft) effective groin length extension only (60 m/~195 ft length weir remaining)
- **Case 5:** 60 m (~195 ft) effective groin length extension only (50 m/~165 ft length weir remaining)
- **Case 6:** 70 m (~230 ft) effective groin length extension only (40 m/~130 ft length weir remaining)
- **Case 7:** 50 m effective groin length extension + 30 m long rock breakwater spur (60 m/~195 ft length weir remaining)
- **Case 8:** 50 m (~165 ft) effective length groin extension + 35 m-long rock south jetty extension (60 m/~195 ft length weir remaining)
- **Case 9:** 50 m effective groin length extension + raised weir elevation (+0.3 m/1 ft) (60 m/~195 ft length weir remaining)

Cases 1–6 were run to assess the optimal effective groin length to maximize benefits to updrift beach accretion (in cross-shore and alongshore direction) and reduce sediment inflow into the inlet, while maintaining some degree of inflow into the sand trap. The groin structure can be divided into two major sections: the "effective" groin section and the "anchor" groin section. The "effective" groin (and "effective length" of the groin) in this study is defined as the measured length of the groin seaward of the MHW line that is the major influencer of hydrodynamics and sediment processes. The "anchor" groin section is landward of the MHW line and is buried and/or has minimal influence on sediment processes on a daily/weekly basis. The "total groin length" includes the "effective" and "anchor" sections.

For clarity, all references to groin length in this document refer to the effective length of the groin.

The anchor groin distance is approximately 45 m/150 ft, however, this is subject to change depending on the designed landward extent of the anchoring armor stone.

Cases 7 and 8 expanded on Case 1–6 by including an additional rock extension elsewhere in the inlet system. The primary goal of Case 7 (breakwater spur addition and groin extension) was to

minimize sediment going offshore and accumulating at the inlet mouth and instead, redirect more sediment into the inlet sand trap and updrift of the proposed extended rock groin. The primary goal of Case 8 (south jetty addition and groin extension) was to drive more sediment accumulation farther east into deeper areas of the sand trap and reduce sediment movement into the inlet throat.

Case 9 was developed to assess a +0.3 m (1 ft) increase in the weir elevation. This +0.3 m increase was for the lower, seaward section of the existing weir. The existing weir and the +0.3 m increase are both within the MHW/MLW tide range, however, the raised weir alternative will reduce the amount of time when water and sediment flow over it. A +0.3 m increase corresponds to the approximate amount of relative SLR expected on the southeast coast of the U.S. by 2050 under an intermediate-low scenario (Sweet et al., 2022). The primary goal of Case 9 was to reduce sediment transport over the weir and increase updrift beach width.

Case 4 was chosen as the base condition for inclusion into Cases 7–9 to account for the effect of the groin (while maintaining sufficient remaining weir length in the model) and to assess its combined function with the additional rock extensions. Cases 7 and 8 assess the relative benefit of additional structures to inlet sedimentation, and Case 9 assesses the relative benefit of raising the weir an additional 0.3 m (1.0 ft).

Figure 5-1 shows a plan view schematic of Cases 1–8 modeled. **Figure 5-2** shows a cross-section view of the modeled groin extensions (including the Case 9 weir elevation adjustment).



Figure 5-1: Proposed Alternatives Plan View

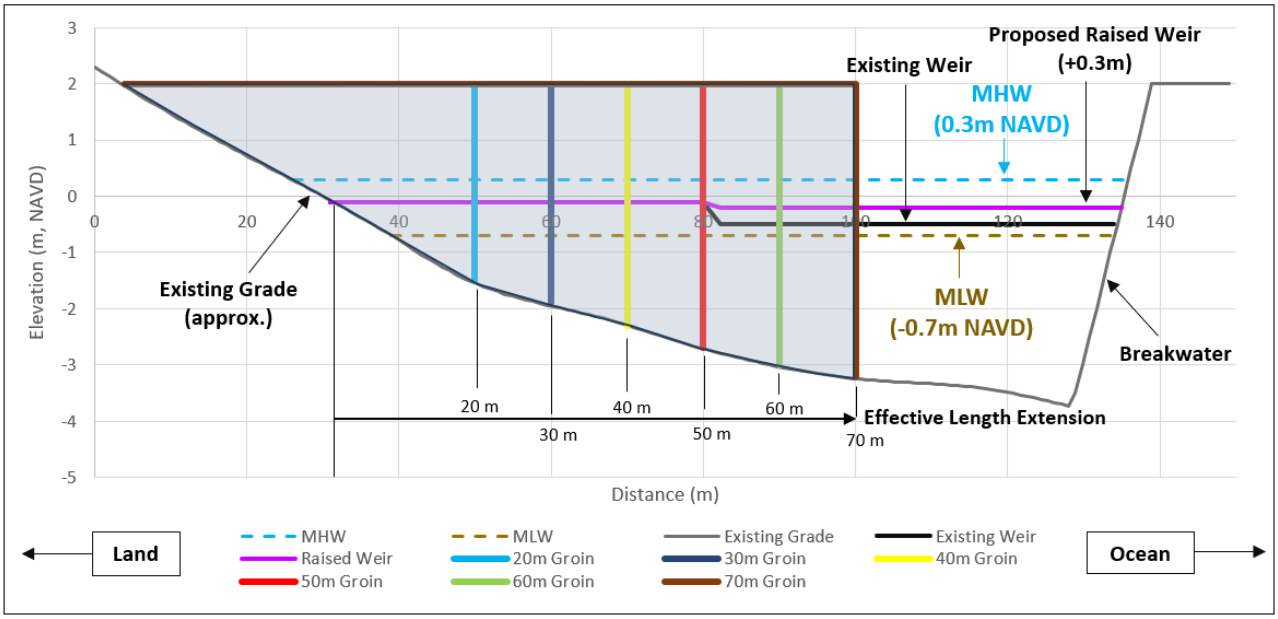


Figure 5-2: Proposed Groin Alternatives Profile View

6. RECOMMENDED ALTERNATIVE

The recommended alternative utilizes the numerical modeling results provided in the previous alternative analysis (Geosyntec Consultants, Inc., 2025) to further develop the conceptual design. The recommended alternative considers multiple options for managing sediment transport and shoaling in the Rudee Inlet sand trap and navigation channel as well as around the breakwater and into the deposition basin. This includes multiple options for improving bypassing methods and increasing dredge efficiency for the City's operations, including the following as shown in **Figure 6-1**.

- A 65 ft weir notch to focus transport into the sand trap;
- Dredge anchors along the sand trap updrift boundary to support efficient dredge operation;
- Restoration and 40 ft extension of the south jetty to limit sand losses from the sand trap into the navigation channel;
- Installation of a south discharge line to provide an alternate discharge location when discharge to the north beach is not practical or feasible; and
- A 250 ft rock groin adjacent to the existing weir to retain more sand on Croatan Beach and improve the performance of the beach nourishment program.



Figure 6-1: Proposed Weir Structure Improvements

The following sections provide additional details for advancing consideration toward the referenced options as part of the recommended design alternative.

6.1 Weir Notch

The weir structure improvements have a primary goal to sustain or improve the City's inlet management operations. This includes maintaining or improving the dredge efficiency for bypassing operations in the sand trap, which provides the last major buffer to capture material prior to entering the inlet channel. The City's dredge operations would prefer to work in Cut 1 of the sand trap, and the proposed groin would encourage shoaling more focused in Cut 2 and into Cut 3.

Results from the numerical modeling analysis indicate the existing shoaling patterns focus towards Cut 1. **Figure 6-2** shows the estimated shoaling patterns for an 8 week period under the assumption all other sediment and restrictions have been removed from the sand trap and inlet channel. The modeling focuses on the shoaling patterns coming over the weir by removing the material volume to the permitted depth in the model set-up.

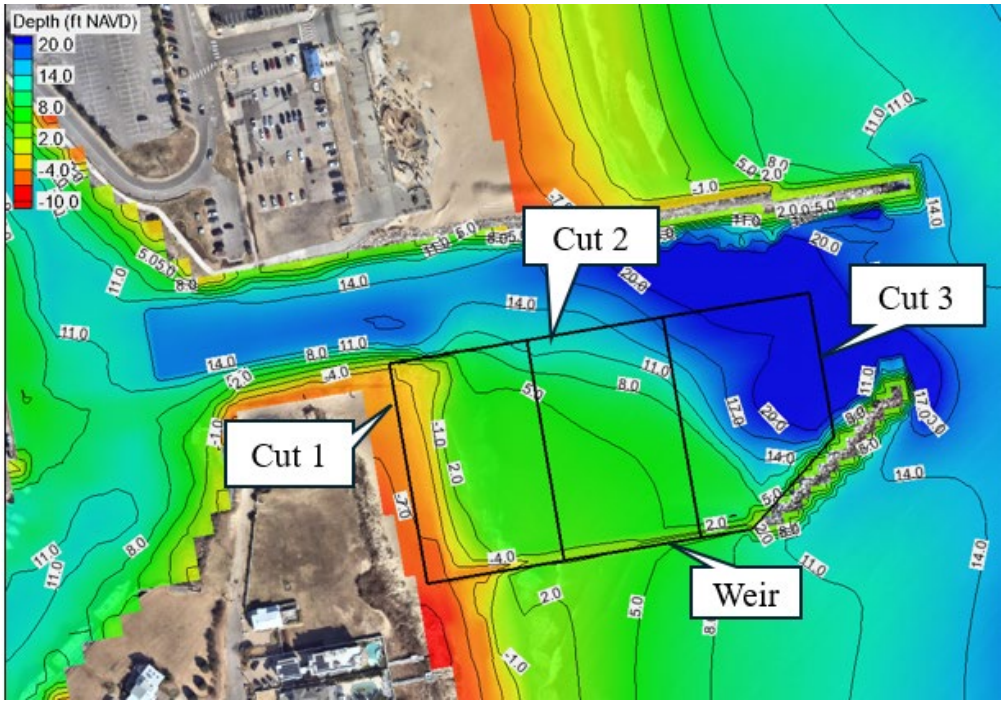


Figure 6-2: Qualitative Example of Baseline Isolated Shoaling Patterns (Initial 8 Week Period)

Comparing the modeled condition of a dredged sand trap under baseline conditions (i.e., conditions without a rock groin or weir notch) with the proposed improvements shows the weir notch will help redirect sediment into Cut 1. **Figure 6-3** shows a color-coded contour map of sediment shoaling advancing into Cut 1 due to the proposed weir notch and groin feature. Although benefits may be observed in the modeling results, adding the weir notch should be further discussed in terms of the construction specifications on how the weir will be cut and in what condition it will be left prior to incorporating the feature in the final design.

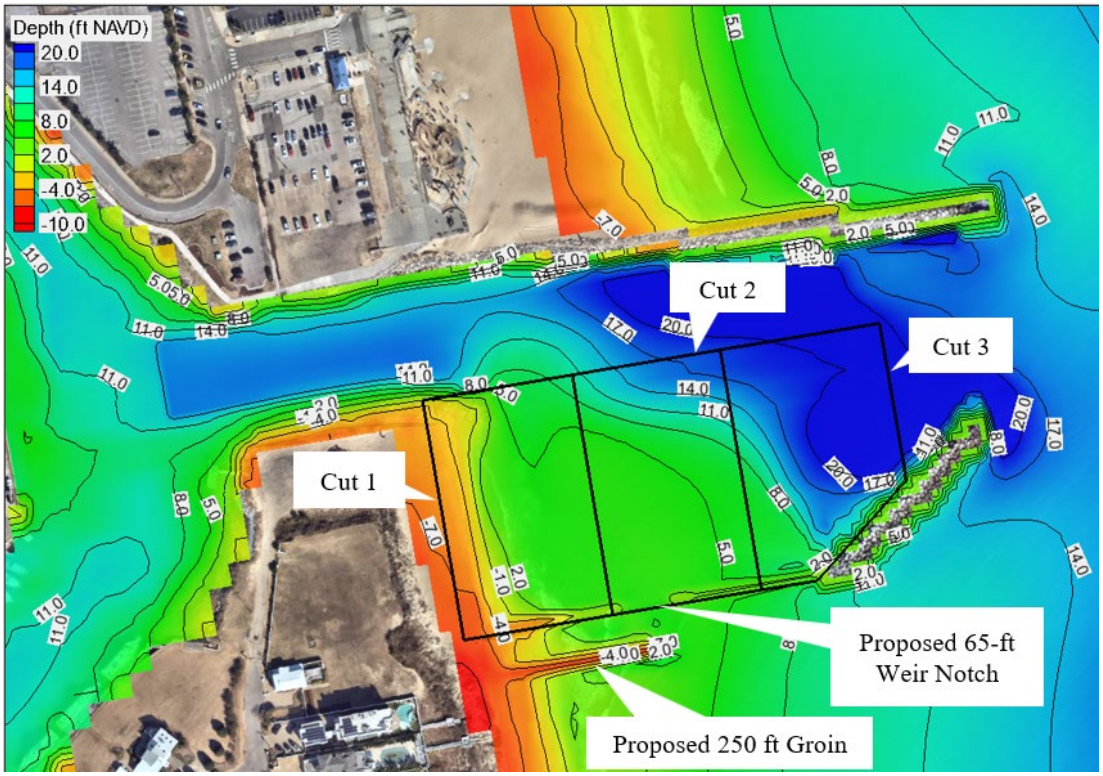


Figure 6-3: Qualitative Example of Isolated Shoaling with the Proposed Groin and Weir Notch

6.2 South Jetty Extension

A secondary element to help improve the sand trap material holding capacity entails restoring the south jetty and extending the feature approximately 40 ft. The south jetty anchors approximately 250 ft of beach between Rudee Inlet and the weir, and improving the structure will benefit the bypassing operations. The proposed improvements include restoring displaced rock along approximately 70 ft of the existing structure and adding new rock to extend the feature 40 ft along the sand trap boundary with the inlet channel. A sheet pile wall will support both the restored section and the extension to protect the City's cutterhead dredge from contacting the rock. The sheet pile wall will encompass the structure and help prevent rock from sliding into the navigation channel or sand trap. The sheet pile wall will be aligned along the slope of the south jetty and will extend to the MLLW elevation (-2.36 ft NAVD). The top crest elevation of the jetty will match the proposed rock groin elevation at +11 ft MLLW (+8.5 ft NAVD), which is 1 ft higher than the existing elevation (Waterway, 2004). **Figure 6-4** shows a plan view of the proposed south jetty improvements.

The modifications will add to the current structure with similar but larger rock sizes. Increasing the rock size from the current configuration will provide additional stability where the smaller rock has partially failed. Rehabilitating the structure will remove or more clearly configure the structure so loose rock material does not interfere or damage the dredge equipment or other watercraft. Extending the structure a maximum of 40 ft seaward will protect a larger sediment cell and add to the existing shoreline.

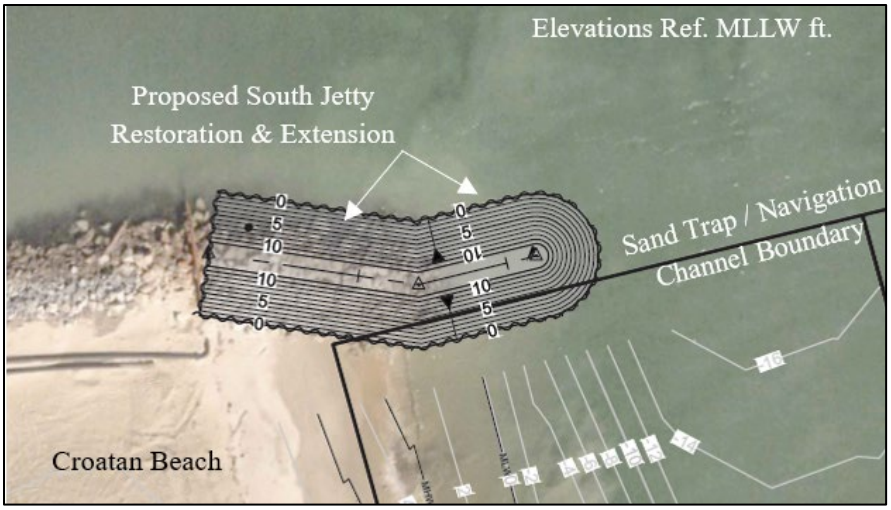


Figure 6-4: Proposed South Jetty Improvements Planview

Modifications to the south jetty will enhance the structure's ability to anchor sediment transport along the shoreline and out of the inlet channel. As sediment migrates through the sand trap and prepares to enter the inlet channel, the south jetty modifications will provide an anchor for holding material on the shoreline. The City's dredge operations will still be able to navigate around the south jetty and focus operations in the sand trap Cut 1. Considerations also provide for potential new dredges or contract plants navigating around the jetty structure, which resulted in the maximum proposed 40 ft length extension.

The south jetty extension should benefit the bypassing operations by holding an additional approximately 4,000 cy out of the navigation channel. The 4,000 cy determination relies on the equilibrium beach profile theory understanding the sand trap maintains a maximum dredge depth of -20 ft MLW (-22.4 NAVD). Material may be expected to accumulate below MHW (0.8 ft NAVD), and the south jetty will allow the shoreline to advance approximately 40 ft seaward. The calculation provides a conservative estimate by assuming only half of the shoreline between the south jetty and the existing weir advances to create the additional storage capacity. The shoreline adjacent to the weir may not accrete due to wave overwash and tidal flows over the lower weir crest discouraging sediment build-up.

Although the south jetty extension will provide additional holding capacity for the sand trap, the City's dredge operations should be confident the improvements will not impact the dredge maneuverability. The City's dredge operations have voiced considerations that the jetty extension would impede their ability to deploy a floating pipeline for transporting bypass material. The City anticipates switching to a new dredge (the Sandpiper) by the end of 2026. During the transfer process, dredge operations should evaluate if the jetty extension makes sense for the new equipment.

6.3 Sand Trap Dredge Anchors

Dredge anchors proposed immediately down drift of the weir and breakwater entail the most significant benefit to enhance the City's bypassing operations. The dredge anchors consist of 24 inch diameter pipe piles installed along the sand trap boundary to provide permanent anchors for the dredging operations. The pipe piles will be installed and filled with concrete to provide fixed

points for swinging the dredge and walking the equipment through the sand trap. The piles will have fixed connections or eyelets so the dredge may quickly connect and continue bypassing operations. Currently, the crew manually positions an anchor in shallow regions of the sand trap to facilitate the dredge's advancement. The crew sets the anchors by dragging them behind a small work boat over the sand trap's shallow waters. Eliminating this process will improve efficiency and reduce the injury risk to workers. With the dredge anchors, the crew will simply need to connect a chain or line to the anchor pile and continue operations.

The pipe piles will be installed on 65 ft centers to provide sufficient anchor points for every situation. The improvements include a total of 16 pipe piles spaced evenly across the sand trap's updrift boundary. The City has already installed similar anchor piles along the north jetty and on both sides of the inlet channel landward of the sand trap. **Figure 6-5** shows the proposed pipe pile layout within the sand trap.

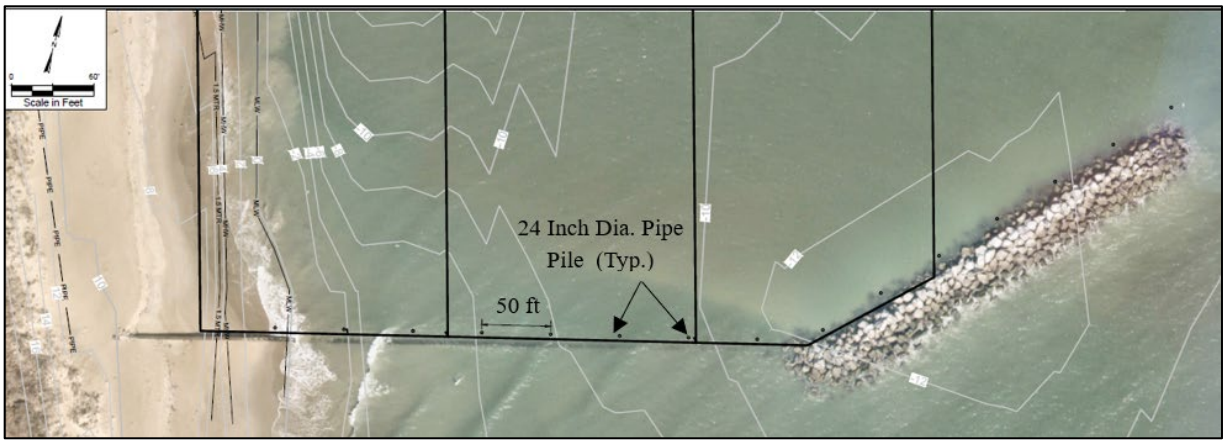


Figure 6-5: Proposed Pipe Pile Dredge Anchors (Plan View)

6.4 South Discharge Pipe

The proposed south discharge pipe will allow bypassing operations to place sediment on Croatan Beach when necessary and provides an alternate placement location when other activities occupy the Resort Beach. The south discharge pipe alternative includes potentially two 18 inch pipelines aligned to carry bypassing material south along Croatan Beach. The discharge lines will rest approximately 6 ft below grade and landward of the +10 ft MLLW (+7.5 ft NAVD) contour to limit the exposure potential during storm events. The pipeline installation will entail directional drilling for placement under any vegetation or dune features. The alternative provides two lines to allow direct access for pumping to designated locations. The lines will terminate at different locations, with one line extending all the way to Camp Pendelton. The second line will terminate as identified by the City.

6.5 250-ft Total Length Rock Groin

The primary benefit of the recommended rock groin focuses on enhancing Croatan Beach. This includes supporting a greater beach width for storm protection and recreation benefits while also extending the intervals for major nourishments. Reducing the weir's cross-sectional opening by constructing a rock groin along the landward stem redirects sediment transport off the shoreline and into the Croatan Beach swash zone. This will create a sediment pocket or fillet where

longshore currents dissipate and encourage sediment accumulation updrift of the groin. As the updrift fillet forms, the Croatan Beach shoreline will extend seaward to realign with the new longshore drift patterns. This process will create a reduction in longshore drift until the shoaling reaches an equilibrium with wave and tidal forces pushing sediment over the weir.

6.5.1 Shoreline & Volumetric Benefit

The 250-ft rock groin alternative will capture longshore transport and add stability to Croatan Beach within approximately 1,200 ft updrift of the weir, or Twilight Lane. (Note, the 250 ft recommended structure maintains a similar length as the 30 m effective length groin discussed in the 2025 alternative analysis.) The captured sediment will create a fillet along the shoreline to increase the Croatan Beach width and extend the waterline seaward. **Figure 6-6** shows an analytical estimation of the shoreline position expected from the fillet in reference to the May 2024 shoreline and overlaid on the project baseline. The analytical analysis, as described by Kamphuis (2002), references the mean tide level as the shoreline location and utilizes additional characteristics as shown in **Table 6-1**.

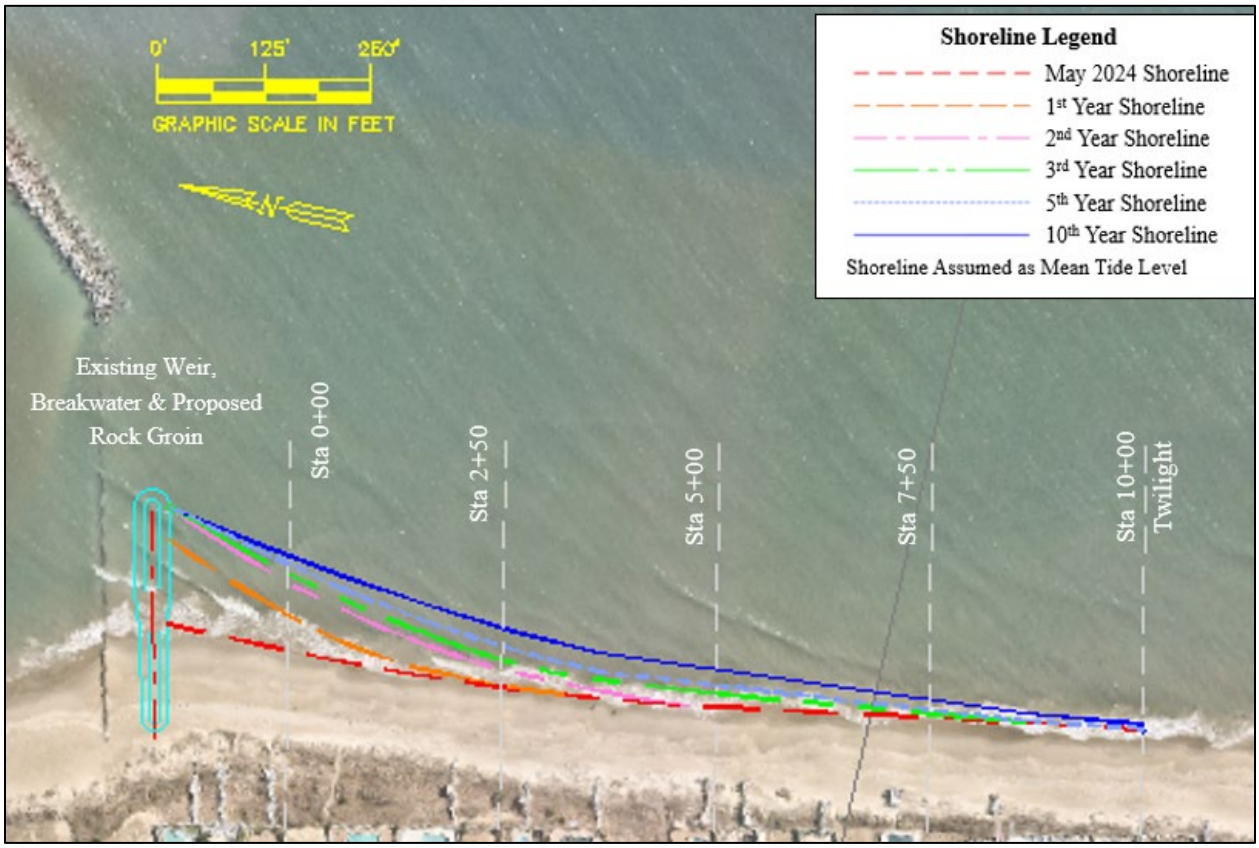


Figure 6-6: Analytical Analysis of Rock Groin 10 Year Shoreline Evolution

Table 6-1: Shoreline Evolution Analysis Constituents

Characteristic	Value	Unit of Measure
Breaking Wave Angle (a_b):	36	°
Sediment Transport (Q):	250,000	cy/yr
Structure Effective Length (S_e):	137	ft
Depth of Closure	-16	ft - NAVD
Berm Height	8.2	ft - NAVD
Mean Tide Level	0	ft - NAVD
Structure Position	Station -1+65	-

The shoreline extension expected from the fillet will provide additional stability and storm protection to the north end of Croatan Beach. The benefit will be observed in terms of a wider beach and additional sand volume within the active profile of the approximate 1,200 ft between Twilight Lane beach access and the existing weir. The benefit will increase over time as sand continues to accumulate within the fillet feature. **Table 6-2** and **Table 6-3** estimate the additional material volume and beach width expected within 10 years following the groin construction. Please note, the estimate does not account for major storm activity but does show the groin structure will benefit Croatan Beach. The estimate utilizes the equilibrium profile theory defined by Dean (2002) to determine the additional profile volume with the depth of closure and berm height as shown in **Table 6-1**.

Table 6-2: Rock Groin Estimated Shoreline and Volumetric Benefit (Baseline Stations -1+65 to 2+50)

Years Post Construction	Sta -1+65 (Groin) Shoreline Advance	Sta -1+65 (Groin) Volumetric Gain	Sta 0+00 Shoreline Advance	Sta 0+00 Volumetric Gain	Sta 2+50 Shoreline Advance	Sta 2+50 Volumetric Gain
1	98 ft	87 cy	24 ft	22 cy	1 ft	1 cy
2	137 ft	122 cy	71 ft	64 cy	15 ft	13 cy
3	137 ft	122 cy	82 ft	73 cy	26 ft	23 cy
5	137 ft	122 cy	94 ft	83 cy	42 ft	38 cy
10	137 ft	122 cy	106 ft	94 cy	65 ft	57 cy

Table 6-3: Rock Groin Estimated Shoreline and Volumetric Benefit (Baseline Stations 5+00 to 10+00)

Years Post Construction	Sta 5+00 Shoreline Advance	Sta 5+00 Volumetric Gain	Sta 7+50 Shoreline Advance	Sta 7+50 Volumetric Gain	Sta 10+00 (Twilight) Shoreline Advance	Sta 10+00 (Twilight) Volumetric Gain
1	0 ft	0 cy	0 ft	0 cy	0 ft	0 cy
2	1 ft	1 cy	0 ft	0 cy	0 ft	0 cy
3	5 ft	4 cy	1 ft	0 cy	0 ft	0 cy
5	14 ft	12 cy	3 ft	3 cy	1 ft	1 cy
10	34 ft	30 cy	15 ft	14 cy	6 ft	5 cy

The rock groin will extend seaward approximately 250 ft from approximately the same landward position as the existing weir and approximately 137 ft seaward of the May 2024 shoreline. The structure's seaward edge will remain within the active profile region to allow sediment bypassing around the seaward tip. The minimized length compared to other alternatives presented in the 2025 alternative analysis allows bypassing and wave actions to continue in a similar fashion as currently observed. The minimized length will reduce the structure's impact on tidal flows traveling over the weir and maintain a similar shoaling pattern in the sand trap as current conditions. **Figure 6-7** provides a color-coded contour map showing the change in velocities during a typical flood tide expected with the proposed 250 ft rock groin.

Estimates show the velocities behind the breakwater will remain well below the 3 ft/s scour threshold (Dean & Dalrymple, 2002) with an estimated velocity increase of 0.5 ft/s [0.15 meters per second (m/s)] and a total velocity of approximately 1.2 ft/s (0.4 m/s) (Geosyntec, 2025). However, the increase should be noted. Extending the rock structure beyond the proposed 250 ft length increases the consideration of how tidal flows behind the breakwater would react and adds additional risk to the City's dredge operations.

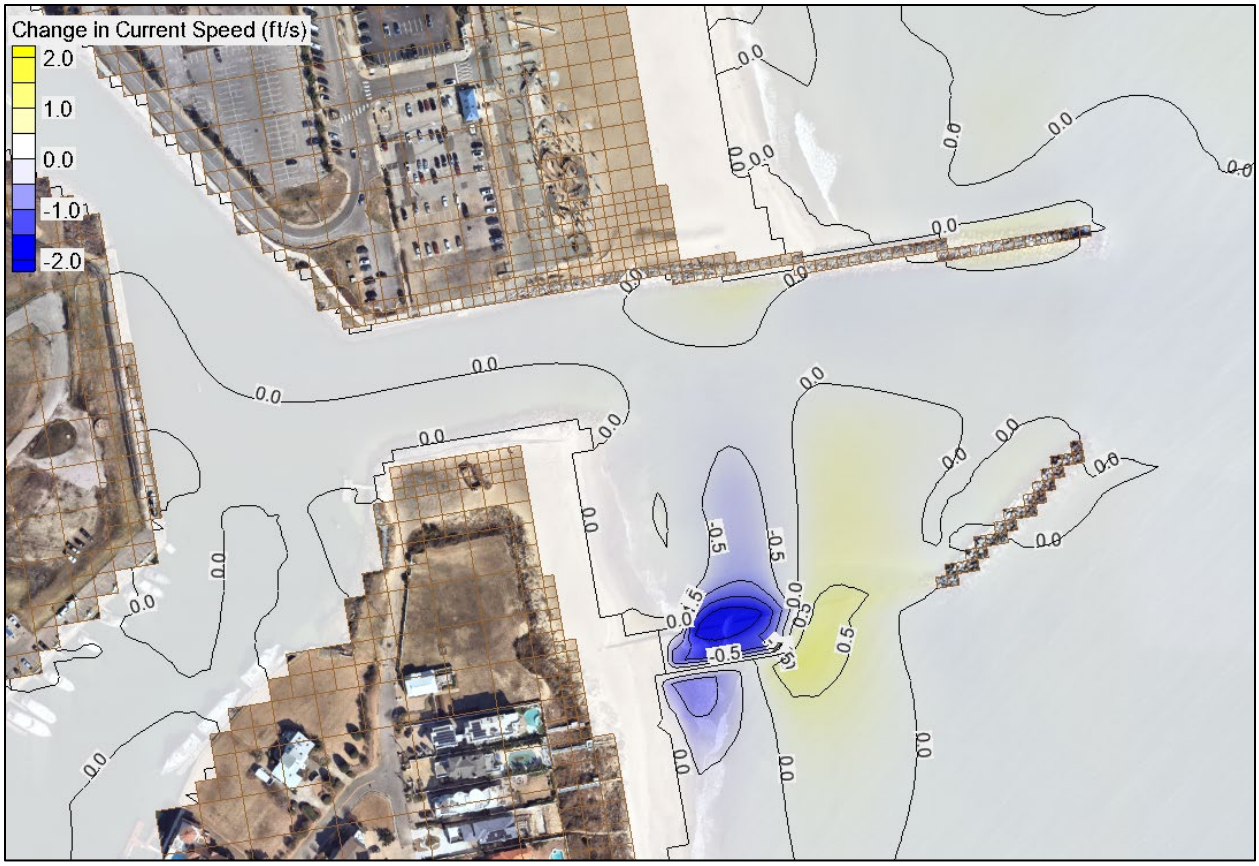


Figure 6-7: Typical Flood Tide Velocity Changes Estimated for the Proposed 250 ft Rock Groin

6.5.2 Sea Level Rise

A significant consideration for any coastal structure revolves around the design resiliency and specifically how the project will respond to SLR. The weir structure improvements provide an opportunity to incorporate SLR through nature-based alternatives. The performance of the weir structure improvements primarily depends on the structure's height. To stop or limit sediment transport, the groin must extend higher than wave transmission to prohibit significant overtopping. (Note, some overtopping during major storms helps buffer crashing wave forces from damaging or moving the rock features.) If the groin extends above the typical elevation waves and tide levels reach, the structure will create a low energy zone and capture sediment transport. Note the structure must also be sand tight so sediment does not pass thorough the voids, but traveling over the structure serves a more direct path for the transport to maintain momentum.

Constructing the groin from rock provides a natural substrate that may be built upon when SLR becomes a significant issue. Additional rock may be placed on top of the groin to extend the height at any future time. This allows construction efforts to maximize the project budget by building in phases and allowing the City to accrue reserve funding for future work. Construction efforts will layer rock in rows across the groin cross-section to build the rubble mound structure. Installing rock later will not hinder the feature's structural integrity. **Figure 6-8** provides an example from the 2004 record drawings of the north jetty showing the layers of a typical groin (Waterway, 2004). Additional layers are not dependent on the existing or design base width and may be added without impacting the structure's stability or performance.

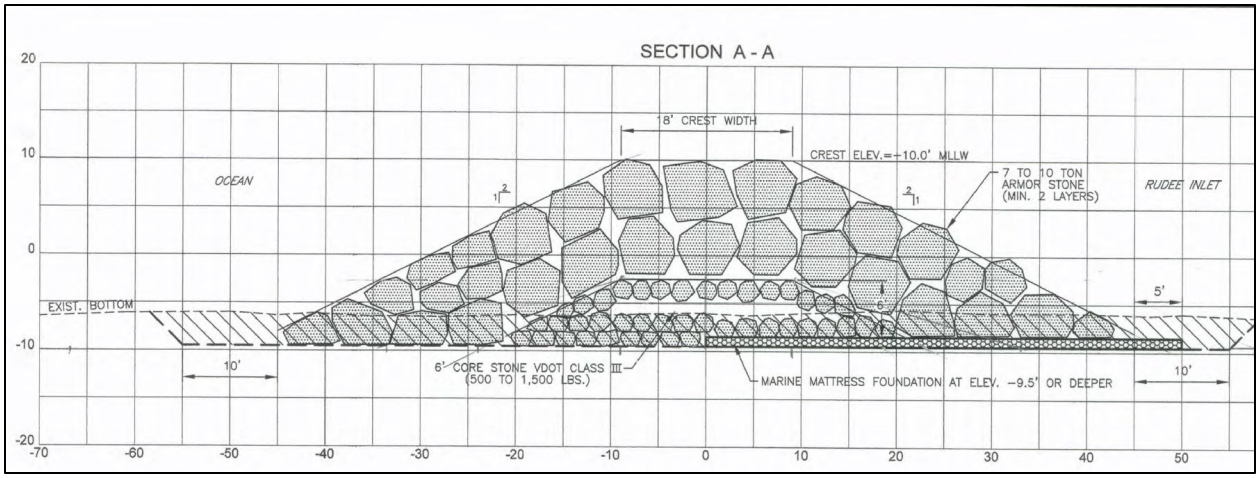


Figure 6-8: Rock Layers in a Typical Groin Cross-Section (Waterway, 2004)

7. ADDITIONAL PROJECT CONSIDERATIONS

As part of the alternative evaluation, the City expressed multiple considerations stakeholders or interested parties had voiced. The considerations relate to the project performance as well as potential social and environmental impacts the construction process may create. The City engaged the Croatan Civic League to help identify the considerations and receive feedback from the local community. The following list provides some of the considerations documented by the City and the Croatan Civic League. Please note, the list may not be all inclusive as new considerations may be brought forward as part of the design review. The City and Geosyntec will continue to discuss and consider additional considerations as they are identified to best serve the communities and user groups of Rudee Inlet, Croatan Beach, Resort Beach, and the vicinity.

- Sediment Transport Rates Around the Breakwater
- Annual Dredge Volumes in the Sand Trap and Inlet Channel
- Shoreline Evolution with the AOC
- Aesthetic Landscape and Construction Activities
- Wave Conditions Supported by the Rock Groin
- Improved Navigation Access Through Rudee Inlet
- Safety and Recreational Space on the Sandy Beach
- Habitat Impacts

The following paragraphs elaborate on how the listed considerations relate to the conceptual design.

7.1 Sediment Transport Rates Around the Breakwater

The City plans and budgets for maintenance events for the inlet channel and outer deposition basin in coordination with the USACE. A variation in the sediment transport rates around the breakwater could significantly impact the maintenance program, especially if the variation occurs with limited or no notice. Therefore, the City requested to include an analysis of the transport rates around the breakwater expected with the proposed weir structure improvements. The analysis focused on potential changes to tidal current patterns, including velocities and directions, that may be created or encouraged from the proposed rock groin and weir notch. The analysis uses the calculated change in current magnitude and direction to correlate the impacts or change in sediment transport. The analysis relies on the modeling work conducted as part of the 2025 alternative analysis for the weir structure improvements (Geosyntec, 2025). **Appendix C** provides the model coefficients chosen for the transport rate evaluation and the 2025 alternative analysis.

Figure 7-1 shows the transport rates will remain consistent with the proposed groin and weir notch. The figure shows the percent change expected in sediment transport around the breakwater and across the weir. The figure also includes the percent change expected in the inlet channel for reference. The analysis found an increase equaling approximately +3% in transport around the breakwater should be expected. Understanding previous studies estimate the net northerly transport to range between 250,000 and 300,000 cy/yr (Waterways, 2001), suggests the transport

already maintains up to a 17% annual variation. Comparing the existing variation of 17% with the additional approximate +3% (approximately 9,000 cy/yr) shows the proposed increase will not create a noticeable change.

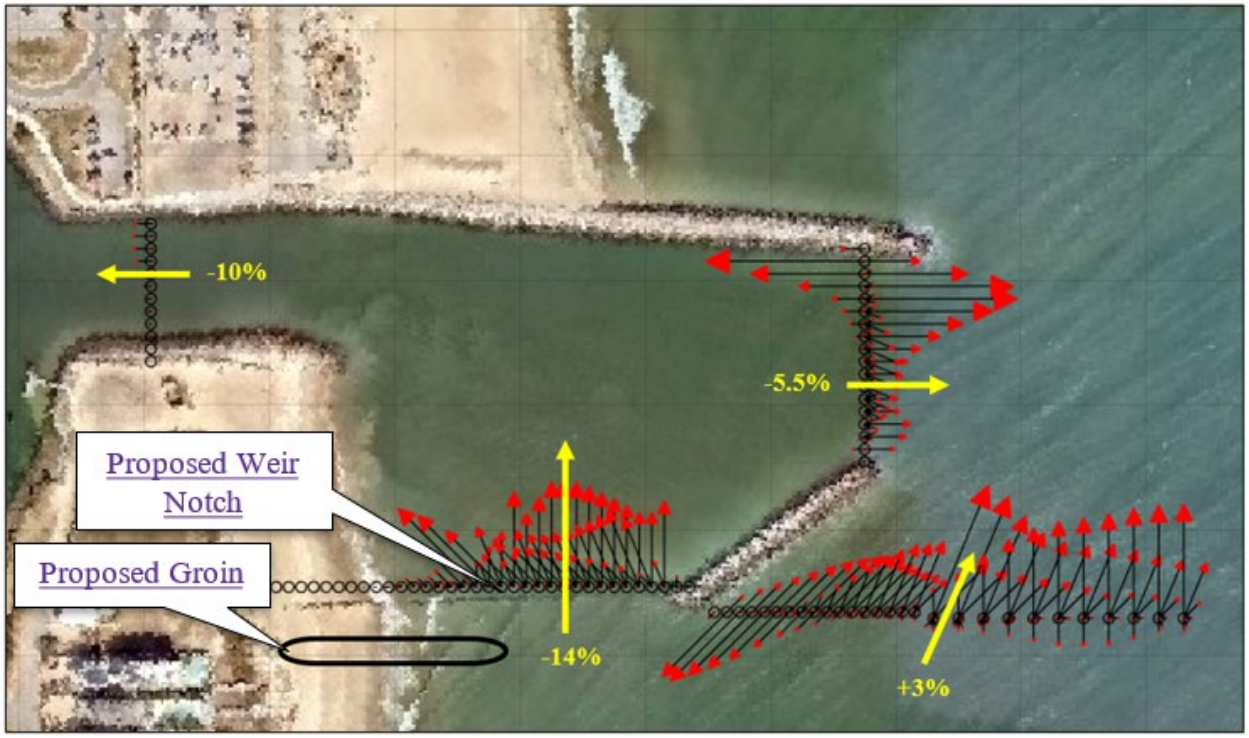


Figure 7-1: Conceptual Design Percent (%) Change in Sediment Transport

Figure 7-1 also shows a decrease in transport across the weir and in the inlet channel. However, the volumetric decrease results from sediment forming the fillet immediately updrift of the proposed rock groin and weir. The decrease in transport should not be considered a project benefit or long-term occurrence, as transport values will return to near existing conditions as the shoreline equilibrates with the new weir structures. The new structure will reduce diffusion losses toward the inlet associated with sand nourishment of the updrift beach. Long term, this will improve the overall performance of the nourishment effort, which may be reflected in a net reduction in transport into the inlet.

7.2 Annual Dredge Volumes in the Sand Trap & Inlet Channel

Similar to the outer deposition basin maintenance efforts, changes or variations in the sediment transport for the sand trap and inlet channel require the City's planning and budgeting efforts. Therefore, the analysis included review of the expected transport and sequential maintenance activities for the sand trap and inlet channel with the proposed improvements. However, as discussed above, the transport rates will remain relatively constant with existing conditions within the overall inlet system once the shoreline equilibrates with the rock groin. The equilibration process should complete within the initial 18 months after construction with the initial fillet formation updrift of the rock groin. Small variations in transport will continue as the fillet continues to evolve as shown in **Figure 6-6**; however, the benefits will not be noticeable as a

reduction on maintenance efforts due to the relatively small magnitudes beyond the initial year post construction.

7.3 Shoreline Evolution with the AOC

How the shoreline will perform with both the proposed rock groin and the implementation of a larger nourishment program from the AOC has been further assessed in this study. If the rock groin holds sufficient material between the weir and Twilight Lane access and erosion is further reduced from the beach nourishment template, the City may reduce the nourishment volume or extend the nourishment intervals to better serve the Croatan Beach community. Evaluating the combined groin induced morphology and the beach fill evolution requires a long-term morphologic model (Dean, 2002) and has been approximated with the analysis in this study. The groin will reduce diffusion (end) losses from the nourishment project and thereby reduce the effective erosion rate within the project footprint. Considering the variables which will impact the shoreline performance, such as the AOC and the weir structures' construction schedule and future weather patterns not considered in the modeling, the City should monitor the shoreline position post construction and reevaluate the performance with measured verses modeled data. The monitoring will provide a direct comparison of the shoreline performance with and without the AOC for future consideration in planning the nourishment events.

Figure 7-2 provides the shoreline position approximated for the shoreline evolution with the AOC, and **Table 7-1**, **Table 7-2**, and **Table 7-3** provide the volumetric gain based on equilibrium profile theory. Comparing the results of the groin induced morphology (**Section 6.1**.) with the combined benefits of the groin and beach fill shows the beach fill will accelerate the groin induced morphology and indicates the nourishment template may potentially be shortened for future maintenance events. However, these conclusions should be confirmed during post-construction monitoring.

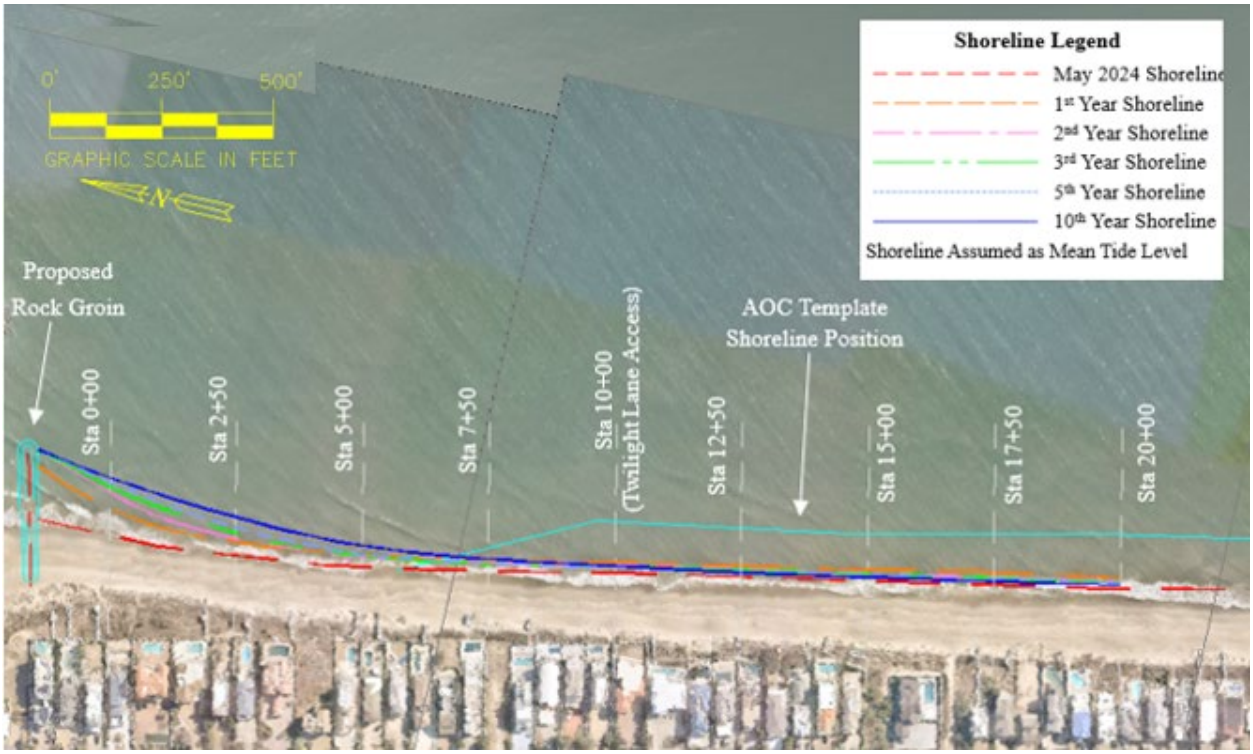


Figure 7-2: Estimated Combined Shoreline Evolution (Proposed Groin and AOC)

Table 7-1: Combined Rock Groin and AOC Estimated Shoreline and Volumetric Benefit (Baseline Stations -1+65 – 2+50)

Years Post Construction	Sta -1+65 (Groin) Shoreline Advance	Sta -1+65 (Groin) Volumetric Gain	Sta 0+00 Shoreline Advance	Sta 0+00 Volumetric Gain	Sta 2+50 Shoreline Advance	Sta 2+50 Volumetric Gain
1	118 ft	105 cy	45 ft	40 cy	21 ft	19 cy
2	137 ft	122 cy	86 ft	77 cy	30 ft	26 cy
3	137 ft	122 cy	95 ft	84 cy	38 ft	34 cy
5	137 ft	122 cy	104 ft	92 cy	52 ft	46 cy
10	137 ft	122 cy	113 ft	100 cy	71 ft	64 cy

Table 7-2: Combined Rock Groin and AOC Estimated Shoreline and Volumetric Benefit (Baseline Stations 5+00 – 10+00)

Years Post Construction	Sta 5+00 Shoreline Advance	Sta 5+00 Volumetric Gain	Sta 7+50 Shoreline Advance	Sta 7+50 Volumetric Gain	Sta 10+00 (Twilight) Shoreline Advance	Sta 10+00 (Twilight) Volumetric Gain
1	21 ft	18 cy	21 ft	18 cy	21 ft	18 cy
2	16 ft	15 cy	15 ft	14 cy	15 ft	14 cy
3	17 ft	15 cy	13 ft	12 cy	13 ft	11 cy
5	24 ft	21 cy	13 ft	12 cy	10 ft	9 cy
10	41 ft	36 cy	22 ft	20 cy	13 ft	11 cy

Table 7-3: Combined Rock Groin and AOC Estimated Shoreline and Volumetric Benefit (Baseline Stations 12+50 – 20+00)

Years Post Construction	Sta 12+50 Shoreline Advance	Sta 12+50 Volumetric Gain	Sta 12+50 Shoreline Advance	Sta 12+50 Volumetric Gain	Sta 17+50 & 20+00 Shoreline Advance	Sta 17+50 & 20+00 Volumetric Gain
1	21 ft	18 cy	22 ft	20 cy	22 ft	20 cy
2	15 ft	14 cy	15 ft	14 cy	15 ft	14 cy
3	13 ft	11 cy	13 ft	11 cy	13 ft	11 cy
5	10 ft	9 cy	10 ft	9 cy	10 ft	9 cy
10	7 ft	6 cy	7 ft	6 cy	7 ft	6 cy

Note, the shoreline evolution results estimate how the AOC diffusion losses combine with the groin induced morphology and show Croatan Beach will receive a significant benefit with the combined efforts. The tables show results through station 20+00 as the midpoint of the AOC template, and diffusion losses further south will mirror the results shown based on the shoreline distance from the project centroid. The results do not account for background erosion or storm impacts. However, the results show the potential benefits available by combining the rock groin with the AOC project.

7.4 Aesthetic Landscape & Construction Activities

The design considers potential aesthetic and construction related impacts for the property owners adjacent to Rudee Inlet. The design incorporates materials similar to the existing weir structures, including the landward base of the weir, the breakwater, and the south jetty. The design acknowledges additional dry beach space will be replaced by the rock groin structure. However, the groin will capture sand and build more planform space in the fillet than the structure will cover. As sand builds along the updrift side, the material will cover the rock, removing it from view. The proposed groin will resemble the north jetty in appearance when fully constructed and in equilibrium with the shoreline.

The construction process will also impact nearby properties as work crews and equipment will be accessing the beach for long periods. Construction equipment and trucks may be expected to be

on the beach for an approximate 4 month period. Large trucks will be transporting rock along the roadways and most likely to the north end of Croatan Beach. Noise impacts will occur; however, the work will be limited to daytime hours and may also be scheduled to avoid weekends. There will be some inconveniences, and the design team will need to communicate closely with the residents (through City staff) and the contractor to minimize impacts before they occur.

Expectations suggest construction efforts will access the beach adjacent to the south jetty, similar to previous dredging projects as shown in **Figure 7-3**. Accessing the beach from the north end will limit interaction between beachgoers and the construction equipment. However, construction equipment will be on the roads longer instead of traveling along the sandy beach. Pending the rock delivery schedule and how many trucks may be expected, the Contractor may be required to travel along the beach, especially with a winter construction period where fewer people should be expected on the beach.



Figure 7-3: Proposed Construction Staging and Access Area

7.5 Wave Conditions Supported by the Rock Groin

The anticipated wave conditions registered as a significant item to address because they directly impact users that will not directly benefit from the project. Croatan Beach locals expressed that surfing near Rudee Inlet provides an important recreational benefit and any impact would impede project success. The wave condition consideration stemmed from the design team's evaluation of incorporating modifications to the existing breakwater. However, the modifications would also change the wave setup dynamics and the resulting wave break. Understanding the consideration to avoid or minimize potential impacts to all user groups, the design team removed the breakwater modifications from future consideration.

Wave conditions also serve as the main reason for selecting the rock groin over replacing or raising the weir structure. The rock materials and void space incorporated in the groin will absorb wave

energy and avoid reflecting wave energy. Raising the weir would create a sheet pile wall and would reflect wave energy back on beachgoers and the shoreline. Similar to a vertical seawall, the reflected wave energy would increase beach erosion and would encourage hazardous conditions for swimmers and surfers. Combining the rock groin with the weir would increase the risk of rocks sliding into the sand trap and impacting the City's dredging operations.

7.6 Improved Navigation Access Through Rudee Inlet

The City and the Croatan Civic League recognize the existing dredging operations create navigation hazards within Rudee Inlet. During dredging operations, the floating line connecting the dredge output to the fixed pipeline located on the north end of Croatan Beach becomes entangled in Rudee Inlet and may block or impede navigation access. **Figure 7-4** shows an example of the floating pipeline in Rudee Inlet. The floating pipeline length cannot easily adjust, so the preferred method to prevent channel blockage requires the dredge to advance into the sand trap. Moving the dredge into the sand trap extends the floating pipeline and helps remove any bends or obstructions in the navigation channel. The weir structure improvements address the navigation considerations by encouraging material to shoal further seaward in sand trap Cut 2. As discussed above, this serves as a benefit to the City's dredging operations and the local recreational boaters.

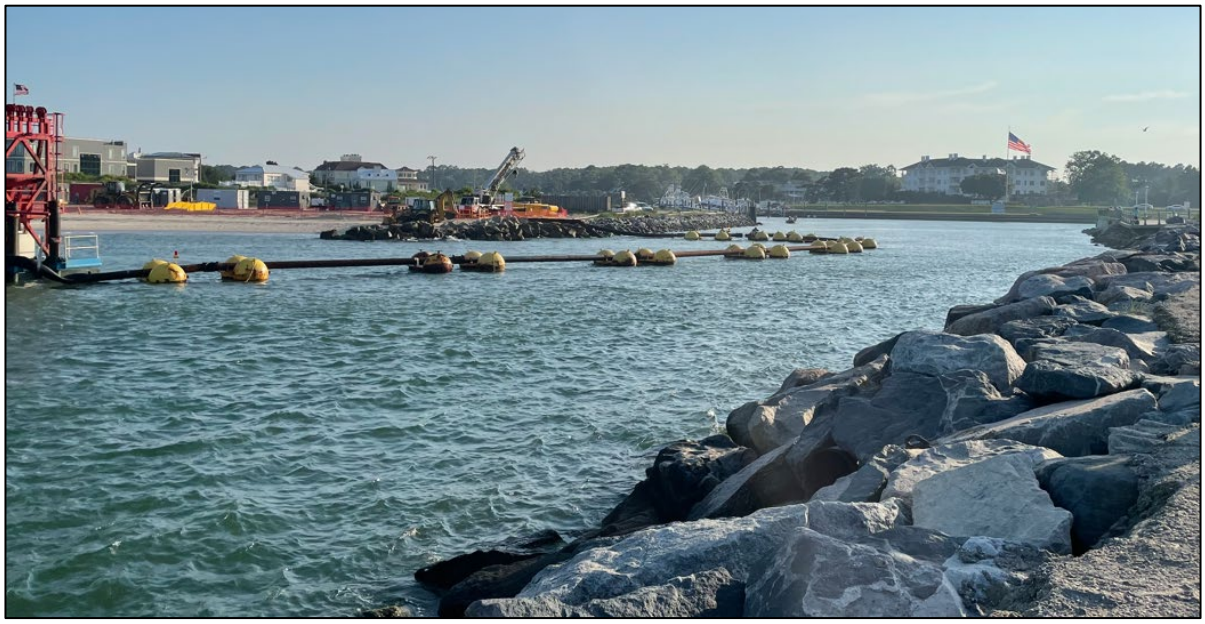


Figure 7-4: Floating Pipeline Positioned in Rudee Inlet

Adding the anchor piles and south jetty improvements will also increase dredging efficiencies and improve navigation conditions. By improving the dredging conditions, material will be removed faster and not have the same opportunity to shoal into the inlet channel.

7.7 Safety & Recreational Space on the Sandy Beach

Understanding the importance of preserving the beach experience for residents and guests, the design team considered multiple alternatives for citing the proposed structures to avoid potential safety considerations. Wave forces or slippery surfaces could create significant impacts for

individuals or animals caught unaware in the void between the structures. Therefore, the design team offset the rock groin to reduce the potential of beach goers incurring injuries. Signage will also be posed to discourage beachgoers from recreating between the structures. During construction, sand will be mechanically placed between the structures to fill the voids and the natural wave action will replenish the material during normal weather conditions. This will limit the potential for negative impacts or injuries and help to preserve the beach experience on Croatan Beach.

7.8 Habitat Impacts

This study also evaluated potential habitat impacts to fisheries in the vicinity of Rudee Inlet. The design team intended to maintain or enhance the available habitat for fisheries and the rock groin accomplishes this goal. The rock groin will provide refuge for juvenile fish through the crevices created by the structure's voids. Juvenile fish will use the crevices for refuge and foraging grounds to enhance the existing habitat provided by the metal weir. Fisheries will still pass over the weir similar to existing conditions since the design will not raise the weir's top elevation or close it completely.

8. CONSTRUCTION SCHEDULE & COST

The estimated construction cost of any project provides a strong influence on the final design. The best design will lose purpose if the budget does not reconcile with the construction means and methods. Geosyntec has worked with the City to maximize the project effectiveness within the available funding expected for the project. The budget for the weir structure improvements equals approximately $4 million as an all-inclusive limit for construction, including any immediate post-construction monitoring and project assessment.

The conceptual construction estimate equals approximately $3,884,190 and should be considered a planning level budget. Geosyntec will continue to refine the estimate during the design and permitting phases as the team considers additional stakeholder comments and unknown permitting requirements that may arise. **Table 8-1** provides a detailed breakout of the conceptual cost estimate.

Table 8-1: Construction Conceptual Cost Estimate

Item	Description	Unit	Unit Price	Quantity	Total Price
1	Mobilization	LS	$410,000	1	$410,000
2	Marine Mattress	LS	$138,000	1	$138,000
3	Sheet Pile Installation	LS	$120,000	1	$120,000
4	Granite Rock	Ton	$85	8,900	$756,500
5	Rock Delivery	Truck Load	$1,125	410	$461,250
6	Groin / Jetty Construction	Day	$5,730	120	$687,600
7	Dredge Anchor Installations	Each	$33,000	16	$528,000
8	South Discharge Dual Line(s)	LF	$32	4,100	$131,200
9	Turbidity Monitoring	Month	$10,000	4	$40,000
10	Clean-up (Demob)	LS	$100,000	1	$105,000
Sub-Total	-	-	-	-	$3,377,550
Contingency (15%)	-	-	-	-	$506,640
Total Conceptual Cost (rounded up for clarity)	-	-	-	-	$3,884,190

Notes:
1. 20 Day construction period estimated.
2. Haul distance estimated at approx. 200 miles.
3. Estimate assumes a 250 ft groin and 40 ft south jetty extension.

The conceptual estimate assumes granite rock will be hauled by truck to the work area and placed by excavator. Access to the beach will most likely occur through the South Atlantic Avenue terminus at Rudee Inlet where the rock may be stockpiled until placed. An offroad truck will transport the stockpiled rock to a construction area designated on the beach for the rock groin and an additional area for the south jetty. Excavators will place the rock extending from the landward-most point seaward. The groin will be constructed updrift of the weir on a marine mattress used to provide foundation support. A marine mattress will also be required for the south jetty extension.

The groin will extend from the beach with approximate 4 ft diameter armor stone covering smaller core stone. As the groin advances seaward, the rock size will increase to account for the higher wave energy expected in the deeper nearshore waters. The average rock diameter will increase from 4 ft to 6 ft between stations 0+90 and 1+15 along the weir. This transitional zone marks the shoreline position used during the modeling evaluation. Station 1+15 along the weir also marks the location the weir crest equals +1.8 ft MLLW (-0.7 ft NAVD). Therefore, the rock groin crest will be approximately 9 ft above the weir opening elevation in the nearshore region. The groin will maintain the average 6 ft diameter rock design throughout the seaward portion for the outer armor stone layer with smaller rock used as core stone within the structure.

The core stone will comprise variable diameter rocks expected to lock together to form a relatively sand tight structure. The mixed diameter rock will adjust naturally through the construction process to fill voids created by larger rocks placed as the foundation. The process will build from the foundation up to the crest at elevation +11.0 ft MLLW (+8.5 ft NAVD), or approximately 1 ft higher than the existing breakwater. The raise in elevation will help discourage overtopping during normal weather conditions and provide an allowance for SLR. The breakwater has performed well and provides an excellent reference for the design criteria appropriate for the project site. Geosyntec will confirm the design meets the physical conditions presented by the anticipated wave climate during the design phase. However, the breakwater provides a reasonable starting point for conceptual planning.

Construction of the groin and south jetty improvements should extend over an approximate 120 day period (4 months), with work limited to weekdays from sunrise to sunset. The work will most likely involve two excavators positioned on the beach with one off-road truck to haul the rock to the respective construction area. A front end loader may also be on site to assist in clean-up and moving the large rocks within the stockpile area. The conceptual design assumes a 250 ft groin length for the design estimate. The proposed structure slopes at 2:1 (horizontal : vertical) and maintains an 18 ft crest width for the 6 ft diameter rock and a 12 ft crest width for the 4 ft diameter rock. The 250 ft groin leaves an approximate 250 ft weir opening accounting for the side slopes in the proposed structure and existing breakwater.

The south jetty improvements will rehabilitate the existing structure and potentially extend the feature approximately 40 ft. Four foot granite rock will be used for the jetty extension and any additional stone needed to rehabilitate the structure. The rehabilitation will consist of repositioning the existing rock to reorganize the jetty feature. Over many years, wave energy has fragmented the jetty and scattered some of the existing rocks. The repair efforts will reorganize the rock and add smaller stone to help lock the larger pieces in place. Further specifics of the south jetty rehabilitation will be confirmed during the design phase, including verification of the appropriate rock size and extension length for the jetty.

9. SUMMARY

The Rudee Inlet weir structure improvements study evaluates alternatives to improve the dredging efficiency for bypassing material in Rudee Inlet and increasing the beach width along Croatan Beach, north of the Twilight Lane beach access. The study evaluates engineering alternatives to anchor longshore transport traveling north from Croatan Beach while also improving bypassing opportunities in the Rudee Inlet sand trap, inlet channel, and outer deposition basin. The study concentrates on improvements to the existing weir system, which has operated since the early 2000's.

The existing weir entails a metal sheetpile structure extending approximately 500 ft from Croatan Beach to the offshore breakwater adjacent to Rudee Inlet. The landward crest elevation starts at +10 ft MLLW (+7.5 ft NAVD) and lowers to +1.0 ft MLLW (-1.5 ft NAVD) at the seaward terminus. The existing sheetpile weir replaced the original wooden structure in 2004 and has served as a key element toward the management of Rudee Inlet and Croatan Beach. Improving the weir system will enhance the ability to capture sand along the weir's updrift boundary. Currently, sediment transport travels across the weir and into the sand trap or inlet channel. A federal study conducted prior to 1999 found Rudee Inlet contributes approximately $3.2 million annually to the local economy. **Figure 9-1** shows a ground level photograph of the weir structure extending seaward toward the offshore breakwater.



Figure 9-1: Rudee Inlet Existing Weir and Breakwater

The City maintains the primary responsibility for managing Rudee Inlet with assistance from the federal and state governments. The City assumed management authority of the inlet in 1989 with the dissolvement of the Virginia Beach Erosion Commission. The City primarily focuses its management efforts toward maintaining the sand trap and inlet channel and receives assistance

from the USACE toward maintaining the outer deposition area. The City maintains and operates a small 14 inch hydraulic dredge (Rudee II) and bypasses an estimated 200,000 cy/yr to the Resort Beach, on the north side of Rudee Inlet. However, previous estimates show the annual bypassing need ranges between 220,000 and 265,000 cy. Rudee II operates within the COLREG lines due to limited maneuverability and works as needed throughout the year. The Rudee II has operated since 2000, and the City will transition to a larger dredge (Sandpiper) by mid-2026. The transition to the new dredge provides the City with an opportunity to review potential improvements to the overall system, including bypassing operations and alternatives to benefit Croatan Beach.

The net northerly sediment transport from Croatan Beach reaches an estimated 250,000 to 300,000 cy/yr. Although the approximately 3,500 ft beach maintains a relatively consistent shoreline, the northern-most 1,000 ft next to Rudee Inlet does not meet the level of storm protection as the remaining beachfront. Replacing or improving the weir system and bypassing operations of Rudee Inlet presents an opportunity to address the shoreline trends within this area of influence from Twilight Lane to Rudee Inlet. The City has conducted annual shoreline monitoring of Croatan Beach along consistent transects since 2017. The monitoring results have aided the City in developing an MBT to meet the protection needs of Croatan Beach. However, the template does not include the area north of Twilight Lane due to the strong tidal currents and wave action that would circumvent any nourishment activity. **Figure 9-2** shows the plan view limits of the MBT on Croatan Beach with the respective area of interest extending from Twilight Lane beach access to the existing weir system.

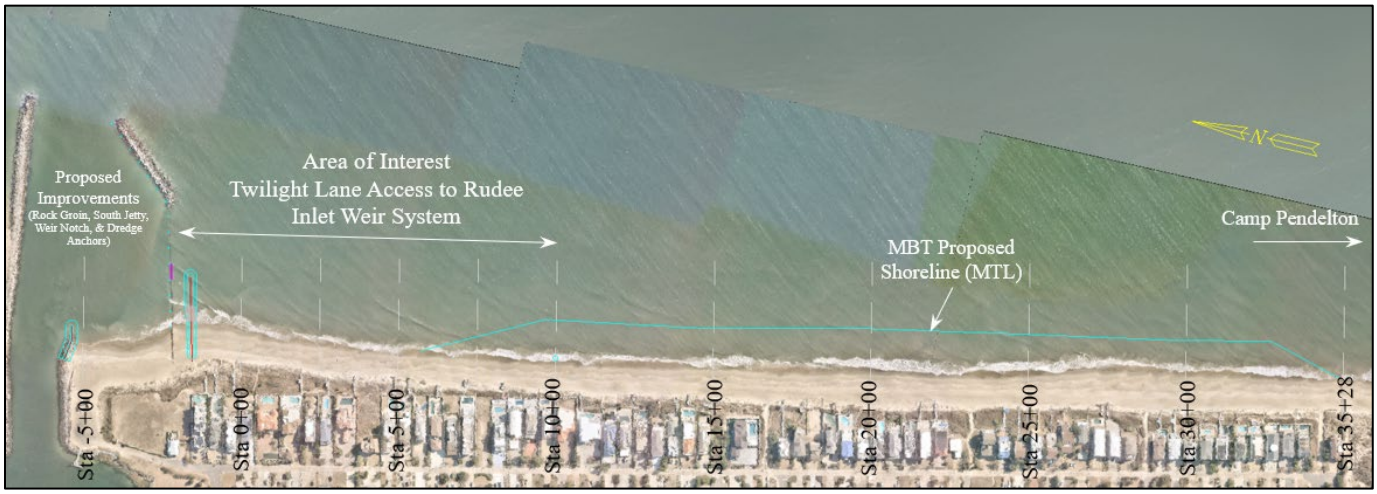


Figure 9-2: Croatan Beach Area of Interest

The study recommends five major improvements to enhance bypassing operations in Rudee Inlet and sand retention along Croatan Beach. The benefits provided by each alternative may be categorized as shoreline or bypassing focused improvements and described further below:

Bypassing Improvements

- A 250 ft rock groin adjacent to the existing weir to retain more sand on Croatan Beach and improve the performance of the beach nourishment program;
- A 65 ft weir notch to focus transport into the sand trap;

- Dredge anchors along the sand trap updrift boundary to support efficient dredge operation;
- Restoration and 40 ft extension of the south jetty to limit sand losses from the sand trap into the navigation channel; and
- Installation of a south discharge line to provide an alternate discharge location when discharge to the north beach is not practical or feasible.

Shoreline Enhancement Alternative(s)

- Rock groin adjacent to the existing weir.

9.1 Bypassing Improvements

The remaining improvements evaluated serve to improve the City's bypassing operations and include the weir notch, the south jetty extension, the dredge anchors, and the south discharge pipe. **Figure 9-3** shows the bypassing improvement alternatives in planview.



Figure 9-3: Proposed Bypassing Improvement Alternatives (Planview)

The weir notch will direct sediment transport into optimal locations within the sand trap for the bypassing operations to capture and transport the material. The notch opens a pathway along the groin's seaward terminus to direct sediment toward Cuts 1 and 2 in the sand trap. Cut 1 generally endures smaller wave forces and provides a more favorable work location due to the sheltering provided by the weir and offshore breakwater. The notch will also offset any impacts to the sediment flow created by the rock groin. The analysis showed the rock groin would increase flood

tidal current velocities behind the breakwater; however, the notch will redirect sedimentation toward Cut 1 in the sand trap.

The analysis used a numeric model scenario to quantify the change in sediment accumulation with the assumption that no other influences applied. The scenario removed all sediment from the sand trap and inlet channel to isolate the weir notches influence and disregard the influence of previous shoaling, abnormal weather patterns, and dredge equipment breakdowns. The modeling provided positive results for the weir notch supported by previous professional experience. However, the analysis should note the scenario represents an isolated condition, and measured results will differ pending the other influences.

The south jetty extension also helps control the sediment flow within the sand trap. Extending the jetty approximately 40 ft along the sand trap northern border should increase the holding capacity by approximately 4,000 cy. The 4,000 cy conservative estimate assumes the extension provides approximately half of the sand trap shoreline, a measurable benefit due to current patterns of wave overwash and tidal flows scouring the shoreline closer to the weir. The estimate also assumes the equilibrium profile theory holds for the shoreline extensions with a profile depth of 22 ft.

Increasing the sand trap holding capacity accomplishes multiple goals. The additional material provides a sediment pocket for dredge operations to target prior to moving into Cut 2 or Cut 3 and keeps the additional material from shoaling into the inlet channel. Both objectives serve as primary goals for the City and this evaluation.

The 40 ft extension includes restoring approximately 70 ft of the structure's eastward terminus and raising the crest elevation to +11 ft MLLW (+8.5 ft NAVD). A sheet pile wall will support the south jetty extension and will be installed with a top elevation equal to MLLW (-2.36 ft NAVD). The sheet pile wall will surround the proposed improvements for additional support during storms or high energy wave events but also will provide a clear transition for the dredging operations. The vertical sheetpile wall will prevent rock from sliding into the sand trap or inlet channel and provide a distinct barrier to protect the dredge equipment.

The dredge anchors and south discharge line also provide direct improvements for dredge operations. The dredge anchors provide fixed connection points along the sand trap boundary to help expedite bypassing operations and improve safety. The anchors entail concrete filled pipe piles installed on 65 ft centers across the southern sand trap boundary. Fixed eyelets installed on top of the piles will allow a direct connection to the dredge with a chain as opposed to crew members dragging a large anchor with the small work boat.

The south discharge line provides options for additional operating times. Periodically, outside activities shut down bypassing to the Resort Beach during festivals or concerts. Bypassing operations may be suspended up to a week or more at a time until the Resort Beach becomes available again. The south discharge line would open Croatan Beach as a bypassing opportunity during the festivals or concerts. The south bypassing lines also provide the City an opportunity to address erosion hotspots on Croatan Beach when needed. Under this proposed alternative, two bypassing lines would be installed to locations identified by the City. The lines would rest 6 ft below grade and landward of the +10 ft MLLW (+7.5 ft NAVD) contour to avoid storm and erosion impacts.

9.2 Shoreline Enhancements

The rock groin serves as the primary feature for enhancing the Croatan Beach shoreline. The groin will anchor sediment along Croatan Beach to provide an increased level of storm protection within the area of interest. The proposed groin will extend approximately 137 ft seaward of the May 2024 shoreline and will maintain 250 ft in total crest length. The crest elevation will extend to +11 ft MLLW (+8.5 ft NAVD), which remains 1 ft higher than the existing weir and breakwater. The increased crest height provides a future benefit for SLR, and the rock material may accommodate additional layers in the future if warranted. Additional rock may be added to the existing substrate as SLR encroaches. This affords the City an opportunity to construct the feature in phases over time, as the structural integrity will support adding future layers.

The shoreline benefit provided by the rock groin will extend to Twilight Lane (Sta. 10+00) but will be considerably more prominent immediately adjacent to the groin (Sta -1+65). With the northerly sediment transport ranging between 250,000 to 300,000 cy/yr, the shoreline adjacent to the groin should advance to the structure's seaward terminus within 2 years. Thus, assuming conditions remain similar to May 2024, the shoreline immediately updrift of the groin will advance approximately 137 ft during the first 2 years after construction. A noticeable benefit will take longer to occur further updrift of the structure, as the fillet continues to evolve over time. The benefit at Twilight Lane will take at least 5 years to form and even at 10 years may only reach a seaward advance of 6 ft, compared to May 2024. **Table 9-1** shows the shoreline advance expected at the fixed monitoring stations between the groin and Twilight Lane from the groin installation.

Table 9-1: Groin Induced Shoreline Advance (Baseline Stations -1+65 to 10+00)

Years Post Construction	Sta -1+65 (Groin)	Sta 0+00	Sta 2+50	Sta 5+00	Sta 7+50	Sta 10+00 (Twilight)
1	98 ft	24 ft	1 ft	0 ft	0 ft	0 ft
2	137 ft	71 ft	15 ft	1 ft	0 ft	0 ft
3	137 ft	82 ft	26 ft	5 ft	1 ft	0 ft
5	137 ft	94 ft	42 ft	14 ft	3 ft	1 ft
10	137 ft	106 ft	65 ft	34 ft	15 ft	6 ft

The shoreline advance will increase the level of storm protection and material volume available to buffer wave energy. The advance will also provide an opportunity to reduce the nourishment volume for the anticipated Croatan Beach nourishments through the AOC maintenance events or to extend the maintenance intervals. **Table 9-2** shows the corresponding volumetric increase associated with the expected groin induced shoreline advance. The volumetric calculations use the equilibrium profile theory to determine the respective quantities with an approximate profile height of 24 ft.

Table 9-2: Groin Induced Volumetric Gain (Baseline Stations -1+65 to 10+00)

Years Post Construction	Sta -1+65 (Groin)	Sta 0+00	Sta 2+50	Sta 5+00	Sta 7+50	Sta 10+00 (Twilight)
1	87 cy	22 cy	1 cy	0 cy	0 cy	0 cy
2	122 cy	64 cy	13 cy	1 cy	0 cy	0 cy
3	122 cy	73 cy	23 cy	4 cy	0 cy	0 cy
5	122 cy	83 cy	38 cy	12 cy	3 cy	1 cy
10	122 cy	94 cy	57 cy	30 cy	14 cy	5 cy

The results of the groin induced morphological changes show that the area of interest between Twilight Lane and the existing weir will benefit with the proposed improvements. Approximately half of the study area will experience a 10 ft seaward advance within 5 years of construction. **Table 9-3** shows the shoreline north of station 5+00 will receive an approximate 14 ft seaward advance or better within 5 years of construction. **Table 9-4** shows the seaward advance should translate to a minimum 12 cy/ft gain between station 5+00 and the proposed rock groin. South of station 5+00, the benefit takes longer to develop but will reach as far south as Twilight Lane (Sta 10+-00) within 10 years of construction. **Figure 9-4** provides a visual representation of the expected benefits.

Table 9-3: Combined Rock Groin and AOC Estimated Shoreline Benefit (Baseline Stations -1+65 to 10+00)

Years Post Construction	Sta -1+65 (Groin)	Sta 0+00	Sta 2+50	Sta 5+00	Sta 7+50	Sta 10+00 (Twilight)
1	118 ft	45 ft	21 ft	21 ft	21 ft	21 ft
2	137 ft	86 ft	30 ft	16 ft	15 ft	15 ft
3	137 ft	95 ft	38 ft	17 ft	13 ft	13 ft
5	137 ft	104 ft	52 ft	24 ft	13 ft	10 ft
10	137 ft	113 ft	71 ft	41 ft	22 ft	13 ft

Table 9-4: Combined Rock Groin and AOC Estimated Volumetric Benefit (Baseline Stations -1+65 to 10+00)

Years Post Construction	Sta -1+65 (Groin)	Sta 0+00	Sta 2+50	Sta 5+00	Sta 7+50	Sta 10+00 (Twilight)
1	105 cy	40 cy	19 cy	18 cy	18 cy	18 cy
2	122 cy	77 cy	26 cy	15 cy	14 cy	14 cy
3	122 cy	84 cy	34 cy	15 cy	12 cy	11 cy
5	122 cy	92 cy	46 cy	21 cy	12 cy	9 cy
10	122 cy	100 cy	64 cy	36 cy	20 cy	11 cy

Additional benefits may be realized by combining the Croatan nourishments with the groin induced shoreline changes. The MBT associated with the Croatan Beach nourishments extends to station 10+00 and tapers north to the existing shoreline. Evaluating the diffusion losses expected from the nourishment shows the project will not significantly benefit the area of interest north of Twilight Lane after the first year post-construction without the proposed rock groin. However, evaluating the diffusion losses with the proposed rock groin shows the shoreline will receive a significant benefit. Excluding background erosion and storm impacts, the area of interest between Twilight Lane and the rock groin will receive a shoreline advance greater than 10 ft within 2 years of construction with the combined benefit of the AOC and the rock groin. The volumetric gain for the shoreline between Twilight Lane and the rock structure equals a minimum of 14 cy within 2 years post construction of both elements.

The analysis estimated the diffusion losses with the assumption the AOC would perform similar to a nourishment on a long straight beach with the rock groin in place. The assumption shows a general 6 ft to 8 ft additional benefit provided by the nourishment project compared to just the benefit expected from the rock groin).

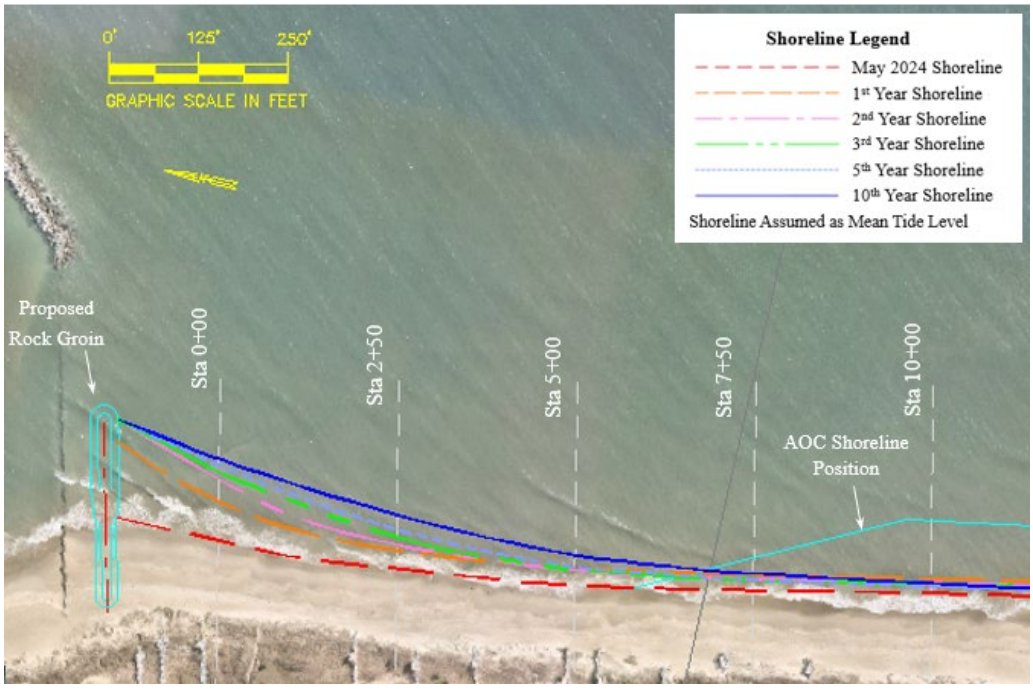


Figure 9-4: Shoreline Evolution with the Rock Groin and AOC (Sta 10+00 to -1+65)

9.3 Additional Considerations

Geosyntec worked with the City to identify and consider a wide range of considerations applicable to the project. The City engaged the Croatan Civic League to identify local stakeholders and solicit input regarding the project. The following list shows the considerations identified as part of the conceptual design process. The design team will continue to address considerations brought forward by the City and stakeholders.

- Sediment Transport Rates Around the Breakwater;

- Annual Dredge Volumes in the Sand Trap and Inlet Channel;
- Aesthetic Landscape and Construction Activities;
- Wave Conditions Supported by the Rock Groin;
- Improved Navigation Access Through Rudee Inlet;
- Safety and Recreational Space on the Sandy Beach; and
- Habitat Impacts.

In efforts to address considerations surrounding potential changes in the sediment transport rates, the analysis recalled the numeric modeling conducted as part of the original alternative analysis in September 2025. The numeric modeling addressed potential changes in the tidal current's direction and magnitude to estimate potential changes in the transport values. The analysis based the general calculations on the observed existing net northerly transport rate ranging between 250,000 and 300,000 cy/yr. The analysis found the transport rates will not experience a significant change under the proposed conditions. The transport rate around the breakwater may increase an expected +3%; however, the documented net northerly transport rate varies by approximately 17% annually. Given the relatively large annual fluctuation in transport rates, the analysis considers the expected 3% change as important to note, but negligible in terms of potential impacts.

Understanding the transport quantities will not change significantly confirms the annual dredge volumes for the sand trap and inlet channel will remain consistent with current practices. The bypassing improvements will increase the dredging efficiency, but the dredging quantities will remain consistent with existing practices.

The design considered the remaining local considerations through best management practices and construction restrictions. The remaining considerations include construction noise and visual impacts, surfing expectations, navigation impacts through Rudee Inlet, beach safety for recreational users, and habitat impacts. **Table 9-5** shows the recommended response for each listed consideration. Although the conceptual design addresses all considerations identified, monitoring should occur through construction to verify the most appropriate corrective action has been implemented. Continued engagement with stakeholders throughout the project is proposed.

Table 9-5: Stakeholder Considerations and Best Management Practice Responses

Consideration	Best Management Practice Response
Construction Noise and Visual Impacts	Limit work hours to daylight and weekdays only. Identify specific access and staging areas so residents and businesses may plan for the event.
Wave Climate Expected	Limit additional structures that may change wave setup features; including a metal sheet pile wall or groin spur.
Navigation Impacts	Redirect sediment transport to allow more work room in the sand trap and outside of the navigable channel.

Consideration	Best Management Practice Response
Beach Safety	Provide adequate room for beach goers. Provide informational signage and markers.
Habitat Impacts	Provide a nature based structure similar to the existing environment with documented benefits / limited impacts.

9.4 Estimated Cost

The conceptual cost estimate for the project equals approximately $3.9 million and is within the $4 million budgeted for this project. The estimate assumes granite rock will be obtained from a source within 200 miles of the site and construction will not extend beyond 4 months. Several items in the conceptual estimate have not been confirmed and will be refined as the design progresses. This includes the sheet pile installation for the south jetty, the dredge anchor piles, and the south discharge lines. Costs have been estimated for these items based on previous professional experience and will be refined as the design clarifies the installation depths, quantities, and type of each element.

The rock groin assumes 6 ft nominal diameter rock will comprise the structure's offshore portion and will transition to 4 ft nominal diameter for the inshore portion. The crest height will be +11 ft MLLW (+8.5 ft NAVD) and will extend a total of 250 ft from the landward origination. The south jetty will be similar with a crest elevation of +11 ft MLLW (+8.5 ft NAVD) and 4 ft nominal diameter stone. The crest height will be 1 ft higher than the surrounding structures, including the weir and offshore breakwater. The increased height will help address SLR by providing an extra 1 ft buffer sufficient to maintain the performance of the structure for an anticipated minimum 50 years of SLR.

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APPENDIX A
September 2025 Numerical Modeling Assessment (Geosyntec, 2025)



Figure 3-1: Plan-view schematic of proposed rock extension (Cases 1-6), rock spur extension off breakwater (Case 7), and south jetty extension (Case 8). Locations of rock placement are approximate. Rock groin extent shown is approximate 50m groin extension length. Overlaid on aerial imagery dated February 2025 (Source: Nearmap.com).

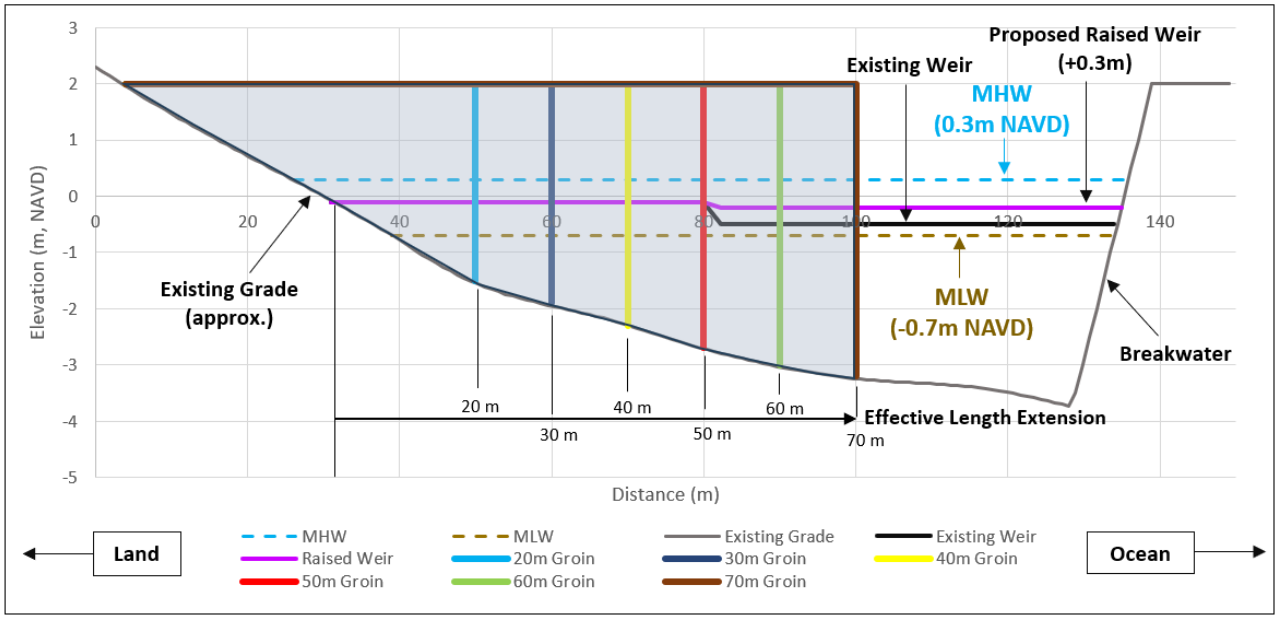


Figure 3-2: Cross-section-view schematic of proposed rock groin extension cases.

4.0 Numerical Modeling Assessment

A numerical modeling study was conducted to provide an objective comparative analysis of the proposed Cases under typical wave and tidal conditions, in the time span of a year. Results allow comparison of changes in inlet sedimentation and shoreline morphology between Baseline Conditions and proposed inlet modifications at the site.

The CMS model suite (CMS-Wave and the CMS-Flow hydrodynamic and sediment transport model) was used for the analysis. The Coastal Modeling System (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.

4.1 CMS Setup

Most model settings were initially set to coefficients and parameters previously used for successful model applications in the region. Sensitivity analysis was also performed for key parameters to qualify model response within the system and determine the most efficient and realistic model configuration.

4.2 Model Grid & Bathymetry

The model grid spanned alongshore from approximately the Resort Beach's 8th street to the north to approximately Camp Pendleton Beach (adjacent to Rifle Range Rd) to the South. In the cross-shore the model grid extended ~1000 m (3280 ft) offshore to ~10m (33 ft) NAVD water depth. Figure 4-1 shows model coverage and depths for the Baseline Conditions run. This grid extent ensured proper coverage of both Lake Rudee and Lake Wesley, marsh and creek tributaries west of Rudee Inlet, and proper wave formation and propagation from offshore to nearshore. Figure 4-2 shows the grid cell sizing used throughout the grid. Model cell sizes were distributed in a telescoping fashion – around the inlet and jetties cell sizes were the most refined at 5 m (~16 feet). Within the nearshore region of the inlet (~2 m to ~6 m water depth) and along the southern oceanfront shoreline, cell sizes were 10m on all sides, and offshore (>8 m water depth) cell sizes increased to 80 m on all sides. Cell sizes were telescoped to maintain high resolution at the inlet while saving model computational time.

Several topographic and bathymetric datasets were merged to create the model grid bathymetry for Baseline Conditions. Table 4-1 lists all datasets used in the present study in order of priority (highest to lowest). Datasets were sourced from Precision Measurements, Inc. (PMI) field surveys, USACE Hydrographic Surveys, NOAA Digital Coast, NOAA Electronic Navigational Charts (ENC), and published Croatan Beach monitoring surveys. Bathymetric and topographic datasets were converted to meters, NAVD88, merged to create a seamless combined dataset, and interpolated onto the CMS model grid (Figure 4-1).

Existing weir elevations were assigned on the model grid based on the PMI bathymetric and ground surveys taken on February 7, 2025, and supplemented with the 2004 Rudee Inlet Infrastructure Improvements As-Builts for the steel sheet pile weir and rock groin construction. The As-Builts were used

to set weir elevations where PMI survey measurements stopped short of the full extent of the weir past wading depth, where elevations are approximately -0.5m NAVD.

Table 4-1: Topographic and bathymetric datasets used for model topobathy (ordered by priority in model interpolation).

Dataset	Source
February 2025 Rudee Inlet bathymetry & ground survey	Precision Measurements, Inc. (PMI)
2004 weir as-built elevations	Waterway Surveys & Engineering
May 2024 Croatan Beach topobathy	Croatan Beach Survey Data
November 2024 inlet bathymetry	USACE Hydrographic Surveys
Lake Rudee and Lake Wesley bathymetry (date unknown)	NOAA Electronic Navigational Charts (ENC)
2019 NOAA LiDAR topobathy	NOAA Digital Coast
Continuously updated Digital Elevation Model (CuDEM) bathymetry	NOAA Digital Coast

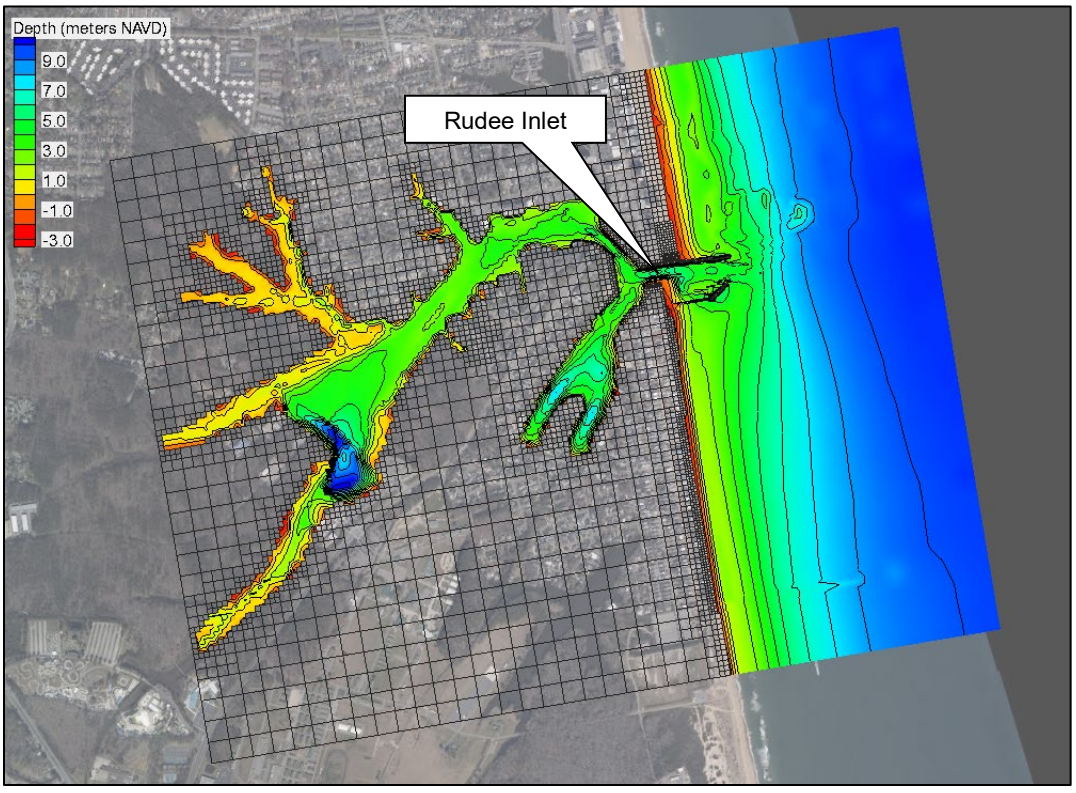


Figure 4-1: Gridded/interpolated bathymetry. Depths are positive and in meters, NAVD.

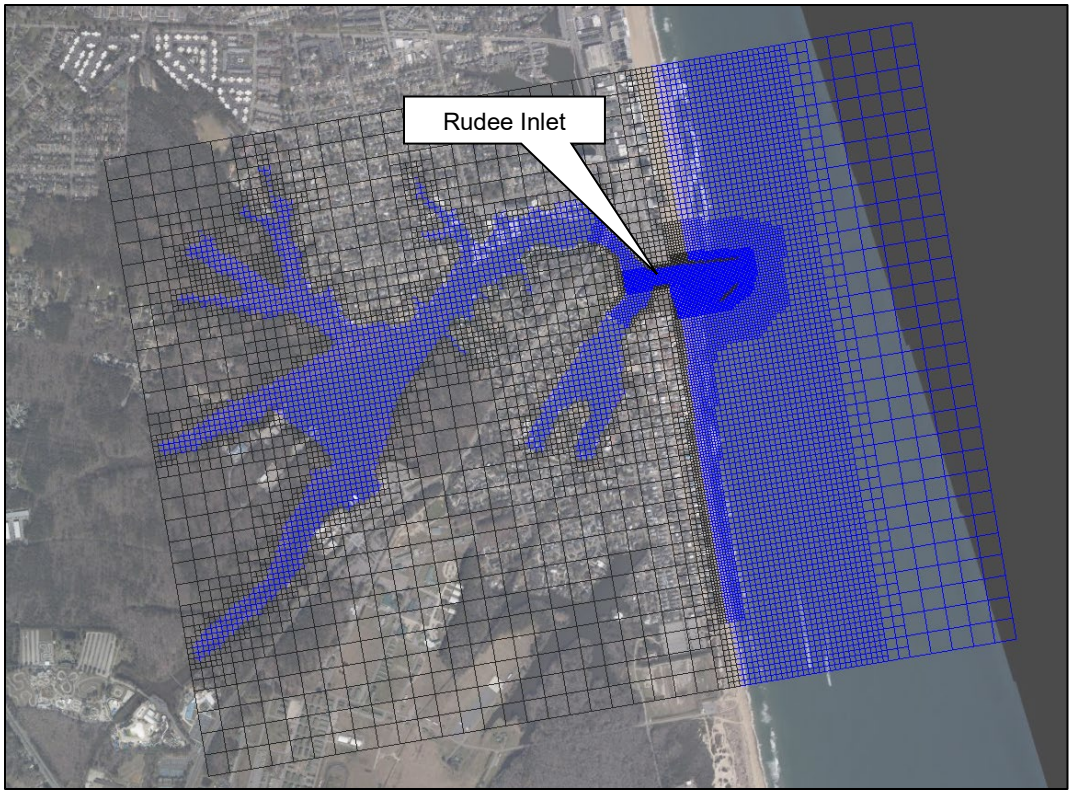


Figure 4-2: CMS-FLOW model domain with telescoping grid cells shown (13,111 total cells).

Two initial bathymetry conditions in the inlet system were considered in this study: 1) a bathymetry grid based on conditions derived from the PMI survey taken on February 7, 2025, and 2) an "After Dredge" (AD) dredge scenario where sections of the inlet system were deepened according to the post-dredge survey conducted by the USACE on April 25 & 27, 2022, which followed the 190,000 cy dredging and beach placement project by Norfolk Dredging. Depths were deepened for sections of the entrance channel, sand trap, and outer deposition areas to -15ft, -20ft, and -22ft MLLW, respectively. This equated to ~9,550 cy of sediment dredged from the main inlet channel, 67,760 cy from the sand trap area, and 121,850 cy from the outer deposition basin. See Figure 4-3 for a comparison of inlet bathymetries between the two scenarios.

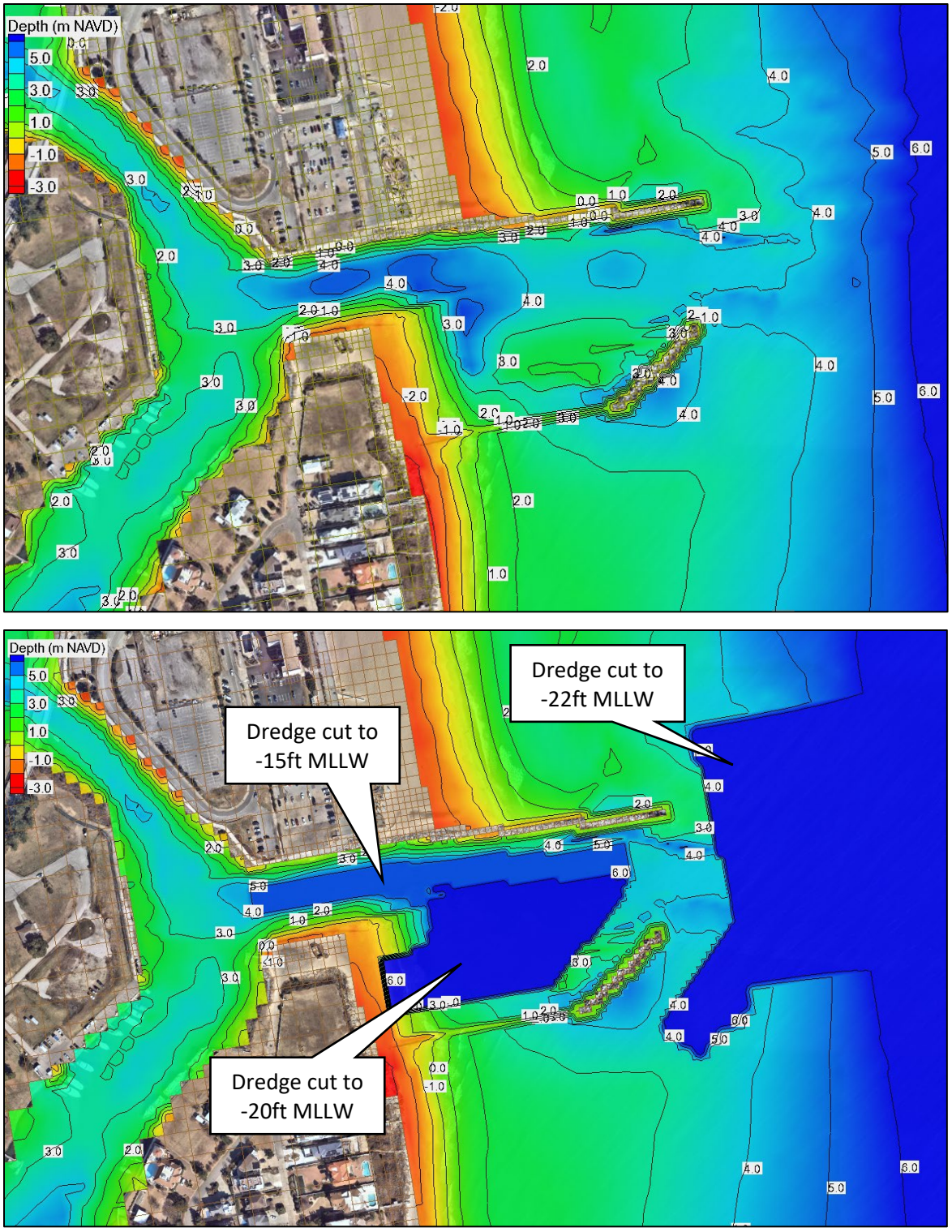


Figure 4-3: PMI bathymetry data (top panel) and "After Dredge" (AD) (bottom panel) model initial bathymetries (depths positive in meters NAVD). Dredge depths in feet MLLW are shown in callouts for reference in bottom panel.

4.2.1 Wave Forcing

WaveWatchIII data was extracted from a point offshore of the project site for the years 2005 to 2019 (the last year WaveWatchIII wave reanalysis production hindcast is available). The offshore extraction point is ~4.5 miles offshore of Rudee Inlet (Figure 4-4).



Figure 4-4: WaveWatch III offshore extraction location.

Figure 4-5 presents a wave rose of the 2005 to 2019 dataset. Waves are primarily directed from the East and Southeast directions, with wave heights typically less than a meter (~3 feet). Figure 4-6 presents a box plot from the 2005 to 2018 wave data set (complete years only). The boxplot is a graphical display that simultaneously describes several important features of a data set, such as center, spread, departure from symmetry, and identification of observations that lie unusually far from the bulk of the data ("outliers"). Wave conditions for the year 2018 were chosen as the primary wave data for model inputs.

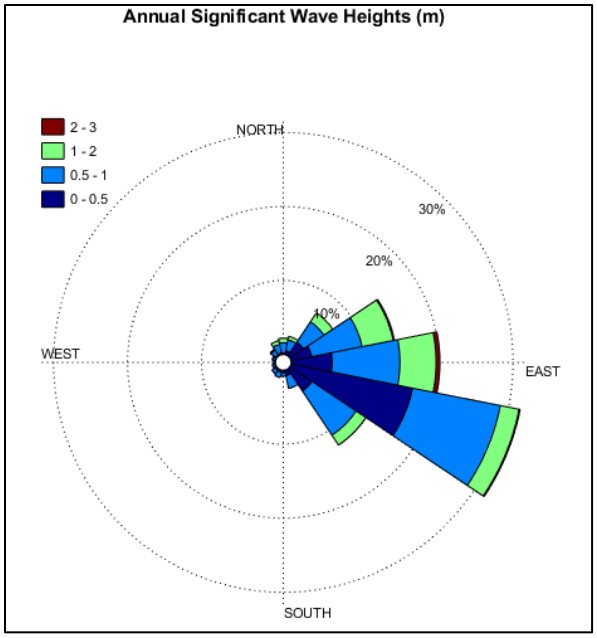


Figure 4-5: WaveWatch III wave rose analysis offshore of Rudee Inlet.

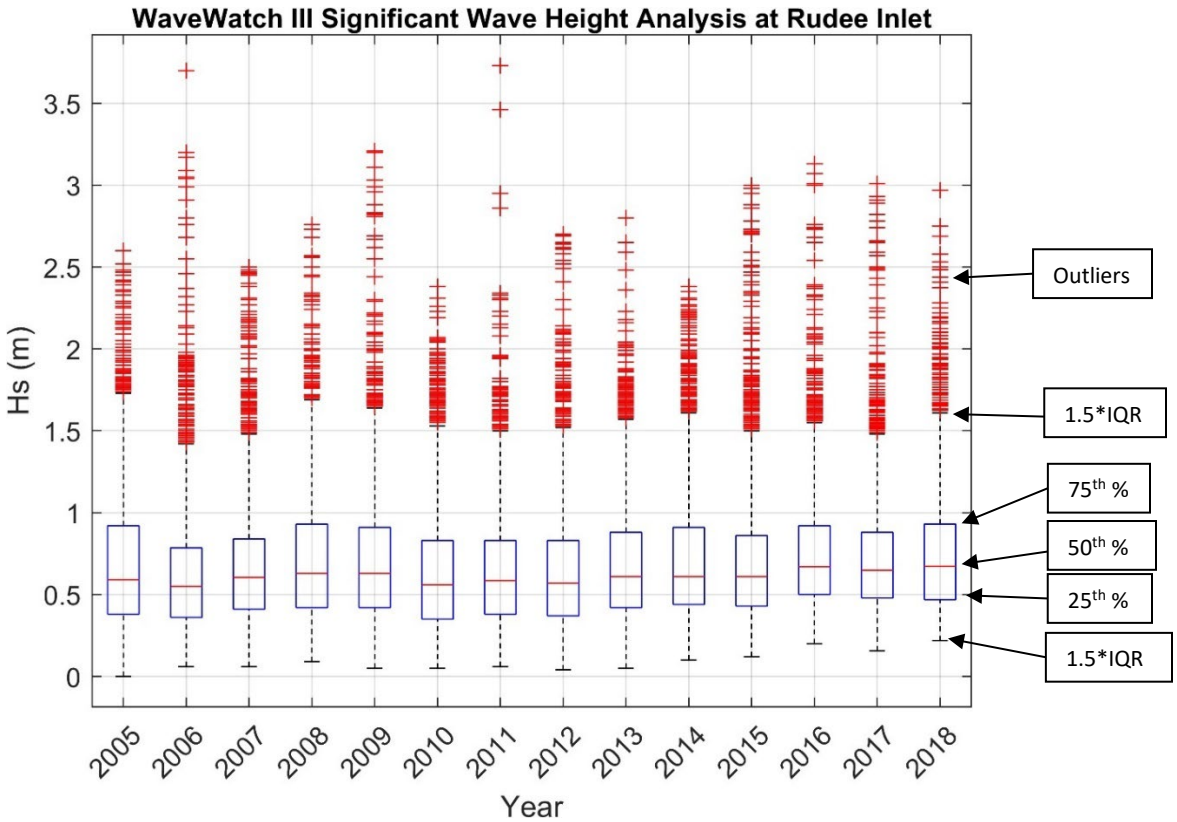


Figure 4-6: WaveWatch III extraction location statistical boxplot. IQR=interquartile range and is the difference between the 75th and 25th percentiles of the data.

Figure 4-7 presents a time series of the 2018 wave heights extracted from WaveWatch III. Figure 4-8 presents a time series of the 2018 wave periods extracted from WaveWatch III.

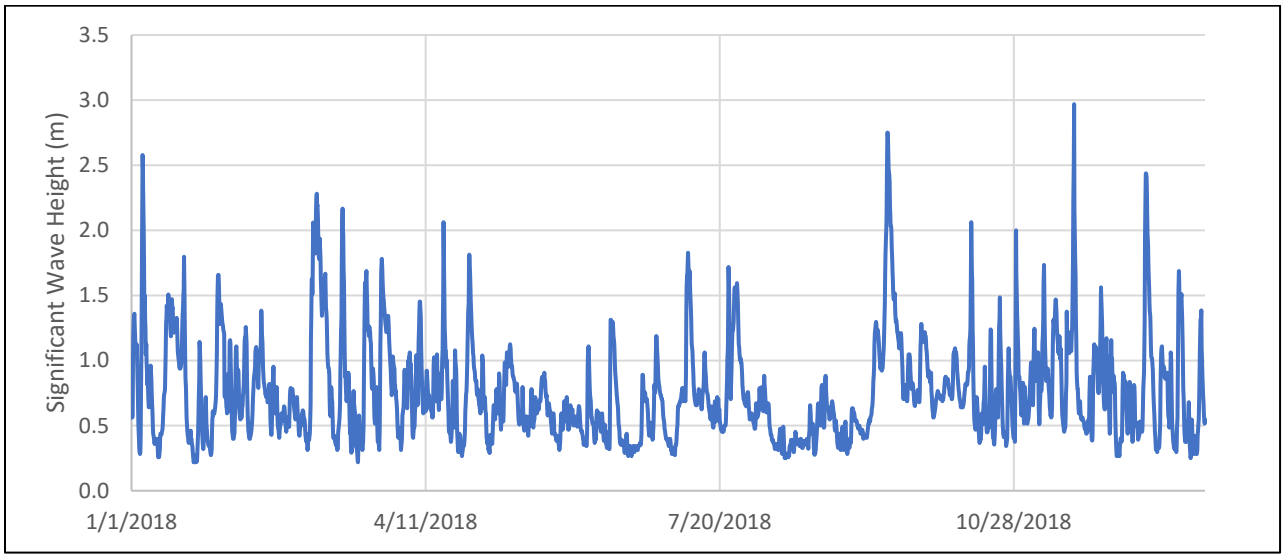


Figure 4-7: Extracted WaveWatch III significant wave height time series for 2018.

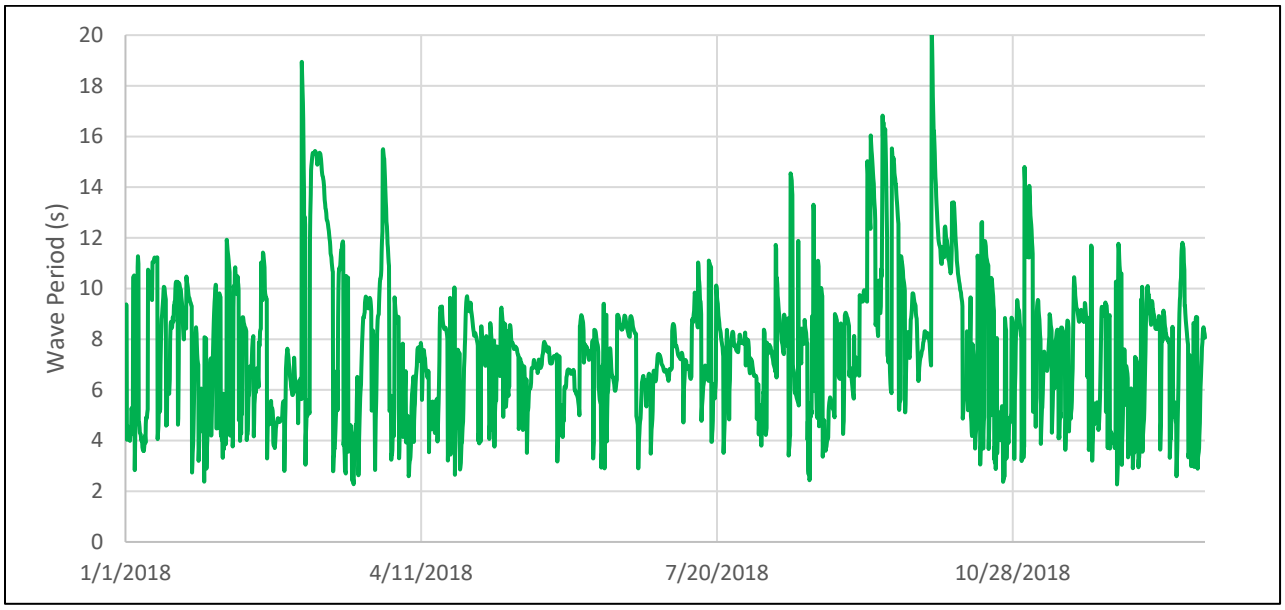


Figure 4-8: Extracted WaveWatch III wave period time series for 2018.

4.2.2 Water Level Forcing

Rudee Inlet currents are mainly associated with tidal fluctuations. Water elevation data was extracted from a nearby USGS Water Level Sensor located in Lake Rudee near Bells Road in Virginia Beach, VA (ID 0204297575). Figure 4-9 presents the location of the USGS gage. Water levels were extracted from the past year, from December 2023 to January 2025 at 6 min increments in feet NAVD and converted to meters for model input. The full year of water level data is plotted in Figure 4-10.

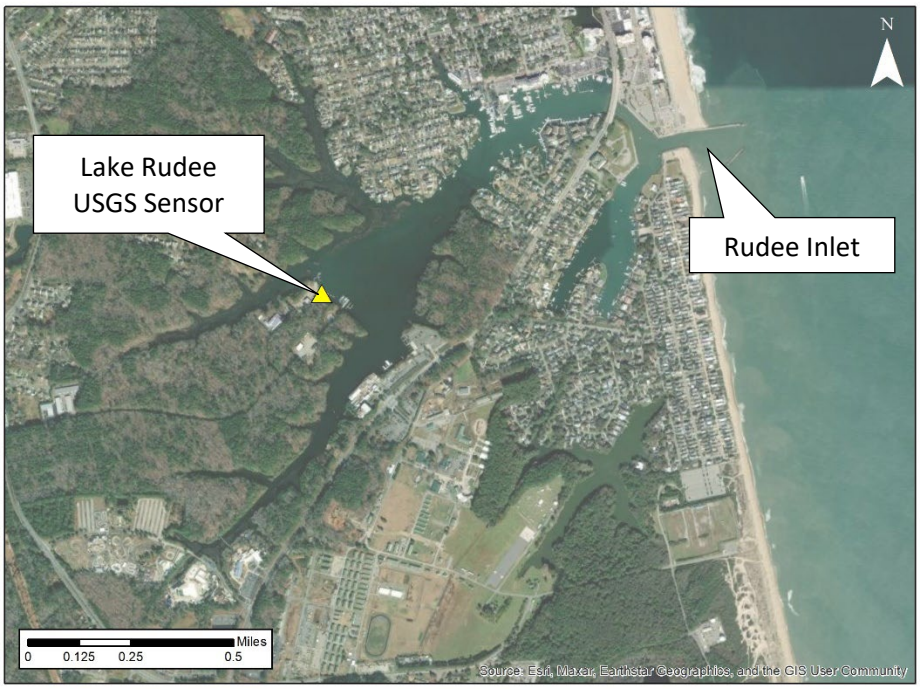


Figure 4-9: Location of USGS Sensor: Lake Rudee near Bells Road at Virginia Beach, VA.

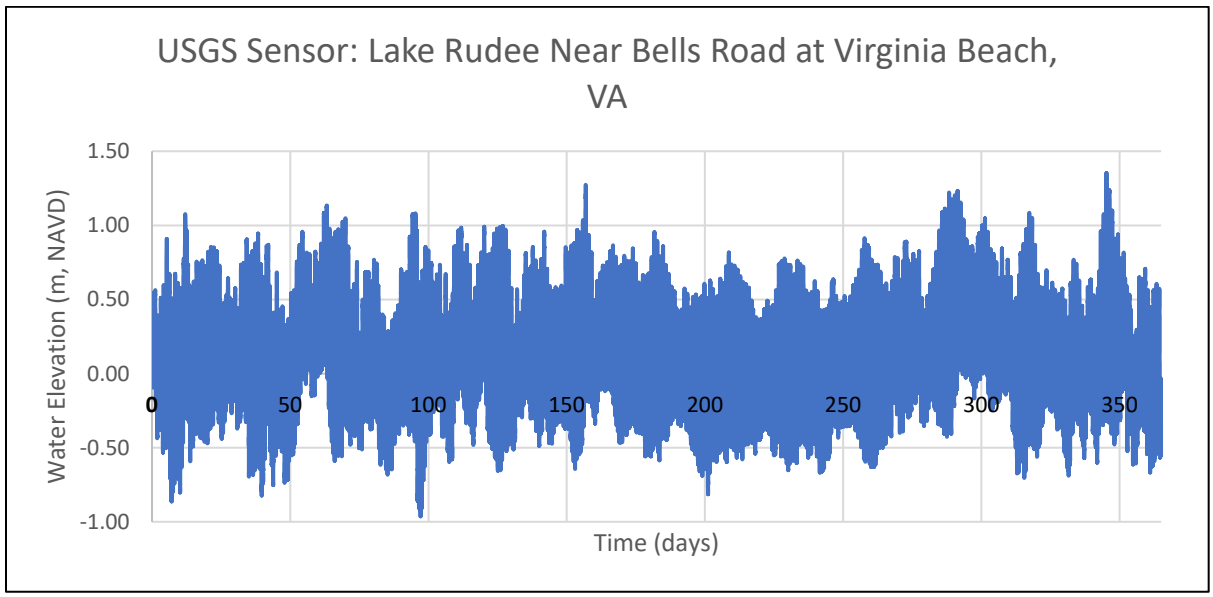


Figure 4-10: Water levels extracted from the USGS Lake Rudee sensor and input into model.

4.3 Model Calibration Methodology

The CMS model was first calibrated to Baseline Conditions at the site. The Baseline Conditions model was run for a one-year simulation under operational wave and tide conditions discussed in Section 4.1.2 and Section 4.1.3.

The Rudee Inlet model was calibrated using 4 primary parameters:

1) Water Elevations
2) Current Speeds
3) Sediment accumulation in inlet system
4) Sediment transport direction and magnitude along the oceanfront

Parameters 1 and 2 compare measured data gathered by Precision Measurements, Inc. (PMI) to modeled data at locations throughout the inlet system. Parameter 3 compares the modeled sediment change to known sediment bypassing/sedimentation rates for the inlet system. Parameter 4 compares net and gross sediment transport to reported rates for the oceanfront.

4.3.1 Water Elevations

Water level measurements were collected by PMI for a one-month period from February 7, 2025 to March 9, 2025 (30 days) on an hourly interval. The water logger was located within the Lake Rudee entrance channel under the Rudee Inlet Bridge. A short 1-month model run was set up where water levels from the USGS Lake Rudee sensor were extracted for the same time period as the PMI survey to force the model from the offshore boundary. Modeled water levels were then compared to measured water levels at the location of the PMI water logger. Tidal phasing and amplitudes at the Rudee Inlet Bridge were compared.

Model data was extracted at the model cell coincident with the Rudee Inlet bridge gauge. Model results reveal excellent agreement with measured water levels, with a root-mean-square error of 0.99. Figure 4-11 shows the measured and modeled water levels plotted together.

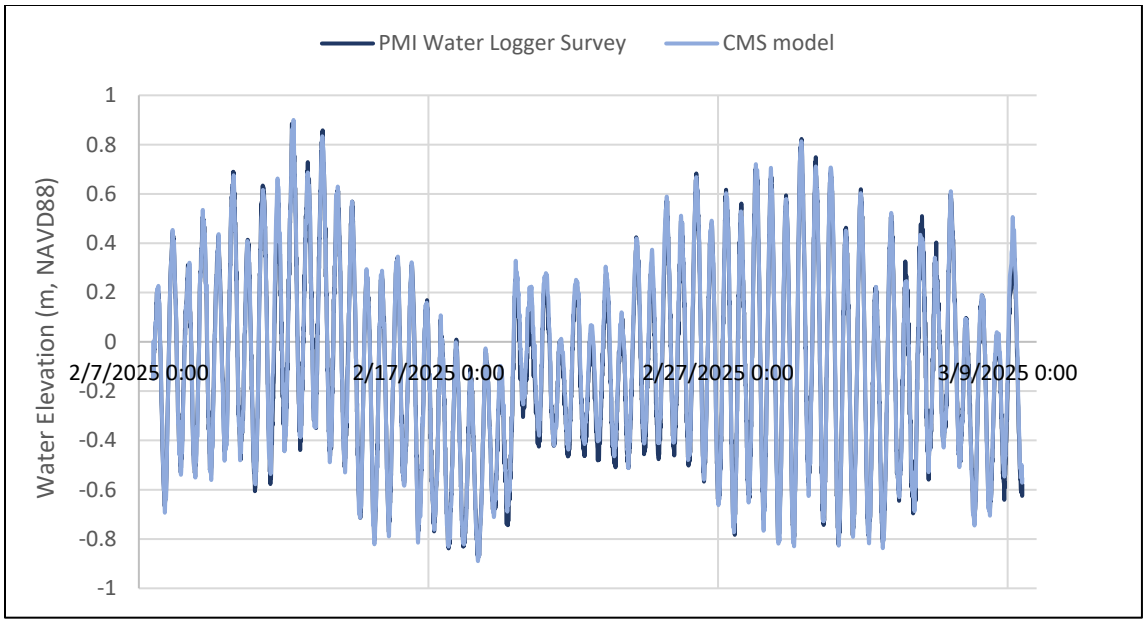


Figure 4-11: Modeled and observed water levels at Rudee Inlet Bridge between February 7, 2025, and March 9, 2025.

4.3.2 Current Speeds

PMI collected current velocity data at each of five locations in the Rudee Inlet System during their field investigation (see Figure 2-4):

1) Rudee Inlet Mouth
2) Rudee Inlet Throat
3) Lake Rudee Entrance
4) Lake Wesley Entrance
5) Weir

Modeled current speeds were compared to observed current speeds for a typical tidal cycle (ebb and flood) to assess model performance. Figure 4-12 shows model snapshots of typical modeled flood and ebb tides for the entire inlet. However, currents throughout the simulation are variable, and may be higher or lower depending on wave and tide conditions.

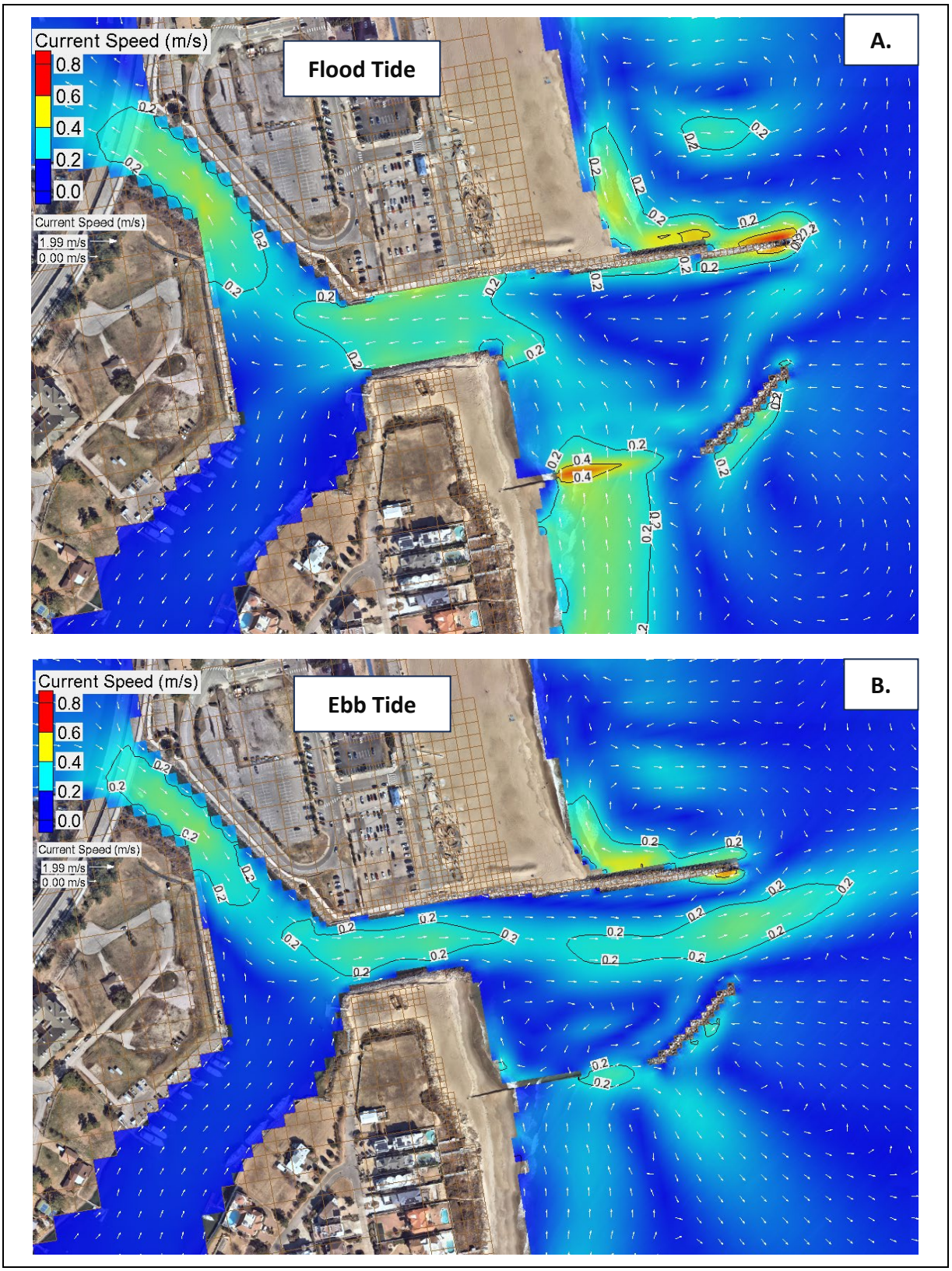


Figure 4-12: Modeled current speeds during typical tidal conditions. A) flood tide; B) ebb tide.

Currents at each of the five field data collection locations were analyzed during the two-week calibration model run (February 7-March 9, 2025) for a single cell at each location on an hourly basis. Figure 4-13 presents a box plot showing the average, interquartile range, and outlier current speeds during the two-week simulation at the approximate center of each of the five transects. Table 4-2 shows the respective value for each statistic by location.

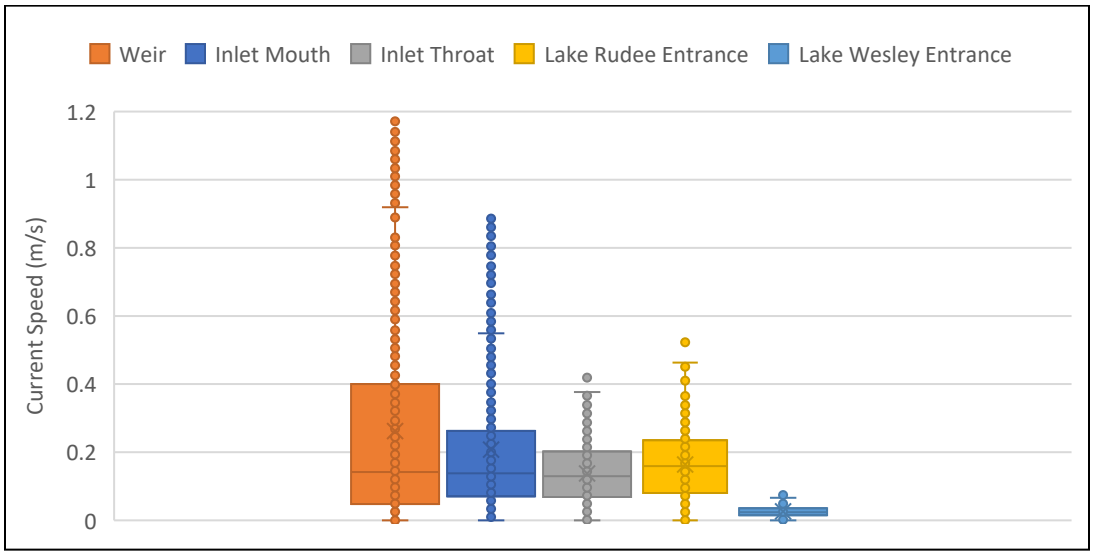


Figure 4-13: Modeled current speed statistics for the 2-Week simulation described in Section 4.2.1 at each of the five locations in the Rudee Inlet System that coincide with the ADCP field collection transect locations.

Table 4-2: Current speed statistics by location for the 2-week simulation described in Section 4.2.1 and plotted in Figure 4-13.

Location	Average Velocity (m/s)	Lower Quartile Velocity (m/s)	Upper Quartile Velocity (m/s)
Weir	0.26	0.05	0.40
Inlet Mouth	0.21	0.07	0.26
Inlet Throat	0.14	0.07	0.20
Lake Rudee Entrance	0.16	0.08	0.23
Lake Wesley Entrance	0.03	0.01	0.04

Results reveal modeled and observed current data show good agreement. Overall, averages are mild/moderate (< 0.3 m/s or ~1.0 ft/s) due to tide fluctuations between high and low tide, which is to be expected. Comparisons between observed and modeled are as follows:

- Modeled current velocities are higher through the Lake Rudee entrance and inlet throat relative to the Lake Wesley channel, which matches trends measured in the field.
- Modeled current speeds through the Lake Rudee entrance, inlet throat, and inlet mouth had upper quartile values within 0.2-0.26 m/s which is in range of the measured average speeds of ~0.2-0.4 m/s.

- Field observations at the weir were taken during an ebb tide, two hours after high tide in the area, and showed current speeds primarily directed out of the inlet up to 0.4 m/s. Measured current speeds are within the quartile range presented in Table 4-2. Though not represented in the field survey, modeled current speeds are higher over the weir during flood tides (similar to that shown in Figure 4-12), due to the alongshore current increasing flow directed south to north.

It is important to note that these current speeds are depth averaged and surface currents, and especially under windy conditions, can be significantly higher (potentially presenting navigation hazards). As a whole, modeled current speeds align well the observed current speeds.

4.3.3 Sediment Transport and Sedimentation

Sediment accumulation in the inlet system was measured after a 1-year simulation and compared to known rates of sediment bypassing/dredging of the inlet. Of note, the inlet sedimentation rates presented in this study are not a direct representation of dredged quantities from the inlet in a given year. The model assumes no dredging efforts throughout the simulation, and represents an idealized case where sediment is allowed to accrete in areas that would normally be dredged. In general, maintenance dredge quantities are greater than anticipated sedimentation rates. Additionally, the USACE sidecaster dredge only pumps dredged material about 100-ft, therefore dredged material remains within the inlet system. Beach nourishments also increase sediment transport, and no Croatan Beach nourishments were modeled for this application.

Several model coefficients/parameters and inputs can be adjusted to increase/decrease modeled sediment transport. A sensitivity analysis was performed to test the various methods and simulate reasonable sediment transport processes. Model calibration focused on both 1) sediment transport and 2) annual sedimentation. Sediment transport and annual sedimentation have the same units (cy/yr) however they are fundamentally different. Sediment transport rates along the oceanfront near Rudee Inlet have been estimated at 200,000 cy/yr to 300,000 cy/yr (Moffatt and Nichol, 2017). The numerical model oceanfront transects exhibit sediment transport rates within this range, noting that sediment transport bifurcates at the weir and offshore breakwater. Annual sedimentation comparisons are described below.

Both initial bathymetry conditions (described in Section 4.2) were considered during model calibration. Figure 4-14 presents the initial and final model topobathy at the site over the 1-year simulation under PMI survey conditions, and Figure 4-15 shows the morphology change (final-initial bathymetry) over the 1-year period under PMI survey conditions. For all model bathymetry change (morphology) figures in this report, yellow indicates accretion/sedimentation (shallower bathymetry) over the 1-year model run while blue indicates erosion (deeper bathymetry) over the 1-year model run. White areas in the morphology change figures indicate negligible changes over the model runs. The solid black lines indicate the boundary in which sediment accumulation was calculated within either the inlet system or the outer deposition area east of the inlet mouth. Under PMI survey conditions, the inlet interior including the sand trap accumulates on the order of 100,000 cy of sediment over the one-year simulation, and the outer deposition area accumulates 30,300 cy of sediment (~130,000 cy combined sedimentation). This is on the order (but in general lower than) recorded sediment quantities dredged and bypassed from the inlet on a yearly basis (assuming no dredging of the inlet exterior and no Croatan Beach nourishments). Based on

this result the sediment transport coefficients have been increased and future simulations for design purposes will utilize the enhanced coefficient parameters (discussed on the following pages).

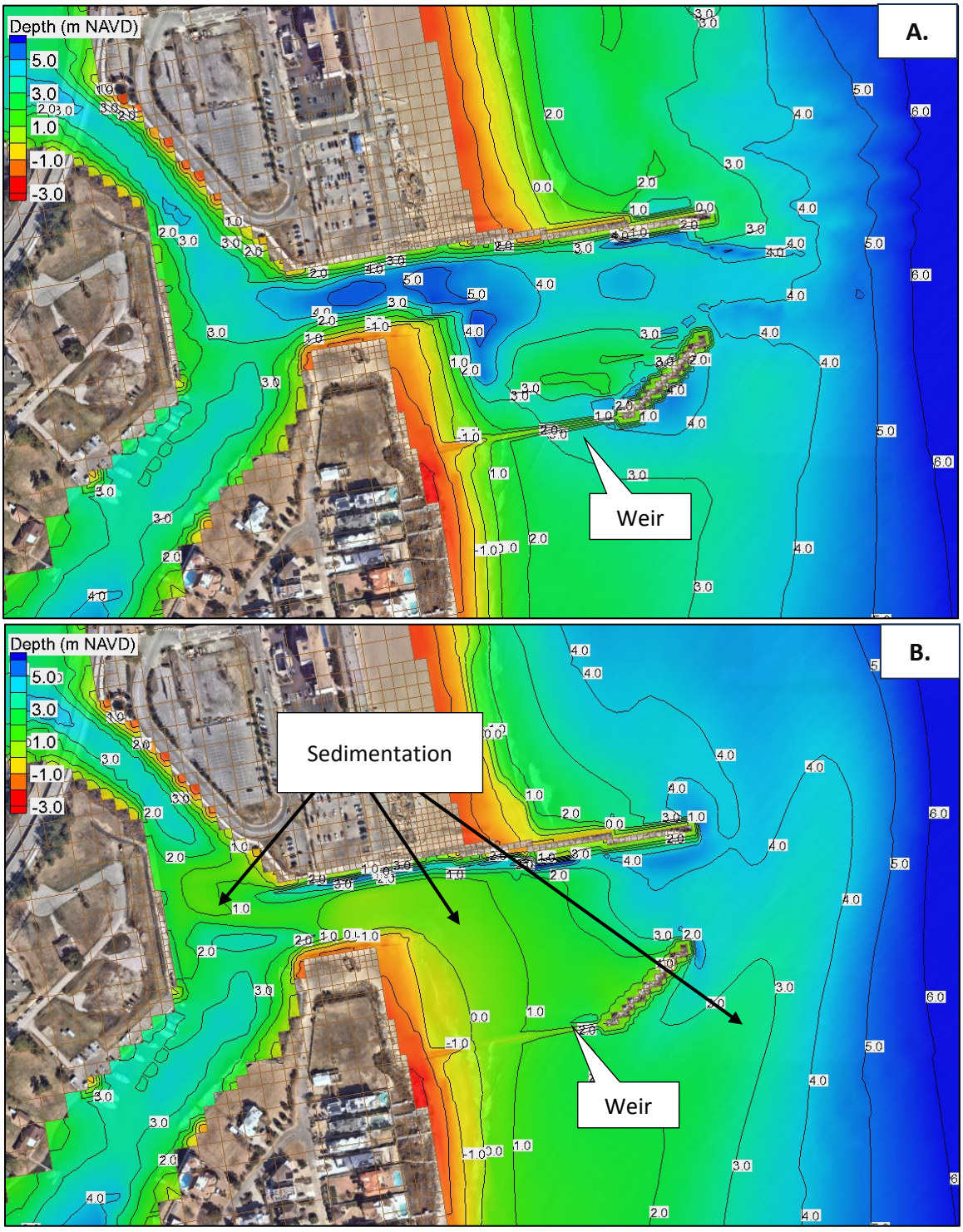


Figure 4-14: Initial and modeled elevations at Rudee Inlet after one year simulation during model calibration under PMI bathymetry conditions. A) initial elevation B) final modeled elevation (both in meters NAVD). Depths are positive.

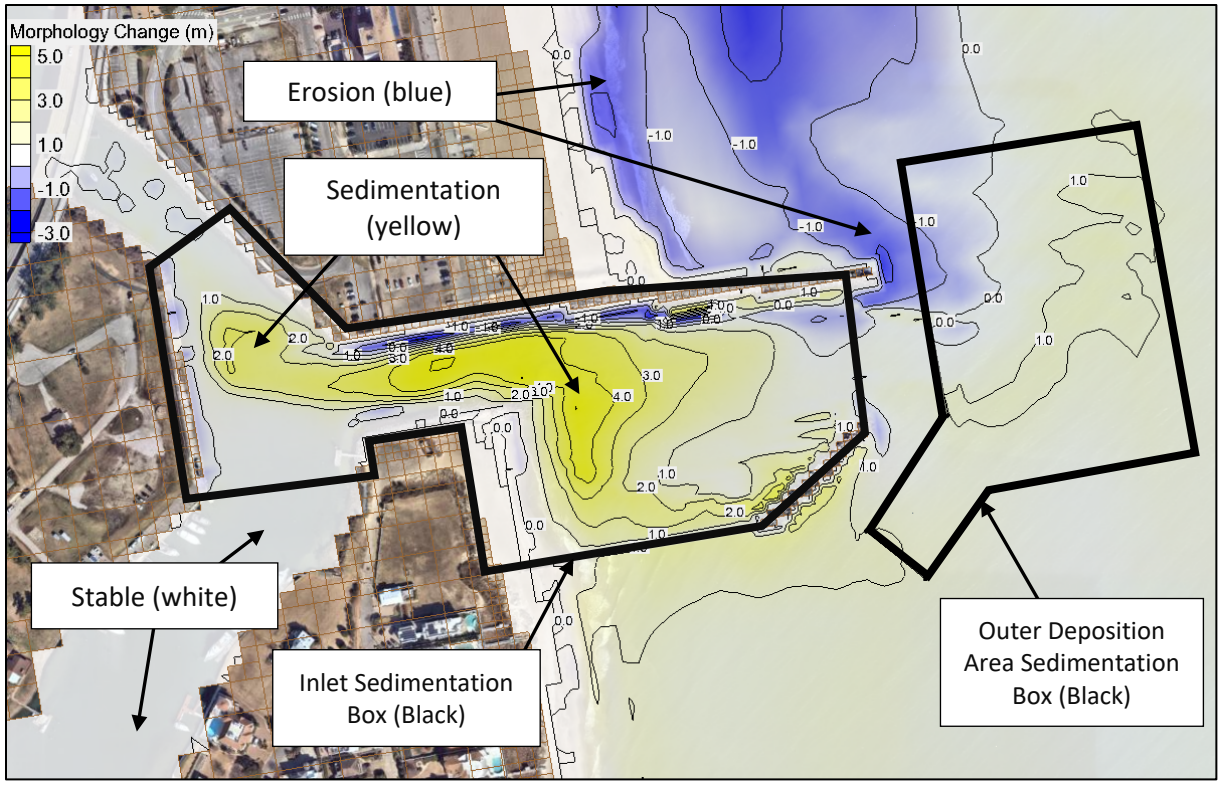


Figure 4-15: Sedimentation and erosion at Rudee Inlet after one year simulation during model calibration under PMI survey conditions. Areas of sedimentation are shown in yellow, and areas of erosion are shown in blue. Areas of negligible change are shown in white. The solid black lines show the areas within which inlet sedimentation was calculated.

Under After Dredge (AD) initial bathymetry conditions, the inlet system accumulates significantly more sediment compared to PMI survey conditions. Figure 4-16 presents the initial and final model topobathy at the site over the 1-year simulation under AD conditions, and Figure 4-17 shows the morphology change (final-initial bathymetry) over the 1-year period under AD conditions. Under AD initial bathymetry conditions, the inlet system accumulated 172,930 cy of sediment, and the outer deposition area accumulated 158,350 cy of sediment over the one-year simulation. In comparison to the PMI conditions run, dredging resulted in an additional +75,080 cy in sedimentation in the inlet system, and +128,050 cy increase in the outer deposition area. As shown in Figure 4-17, sediment accumulation in this run was primarily focused in the dredge cut areas. Results reveal the importance of considering starting model bathymetries in determination of sedimentation rates, and how dredging encourages settlement within the dredge cuts as the system equilibrates.

Certain model parameters were adjusted to increase sediment transport and inlet sedimentation in the model. An additional model run was conducted with sediment transport increased to represent maximum sedimentation in the inlet system. Under these model conditions, the inlet system accumulates ~188,030 cy of sediment, and the outer deposition area accumulated 161,920 cy of sediment over the one-year simulation. In comparison to the AD initial condition run above with model parameters left to default values, ramping up sediment transport led to an additional +15,100 cy of sedimentation in the inlet system, and 3,570 cy increase in the outer deposition area.

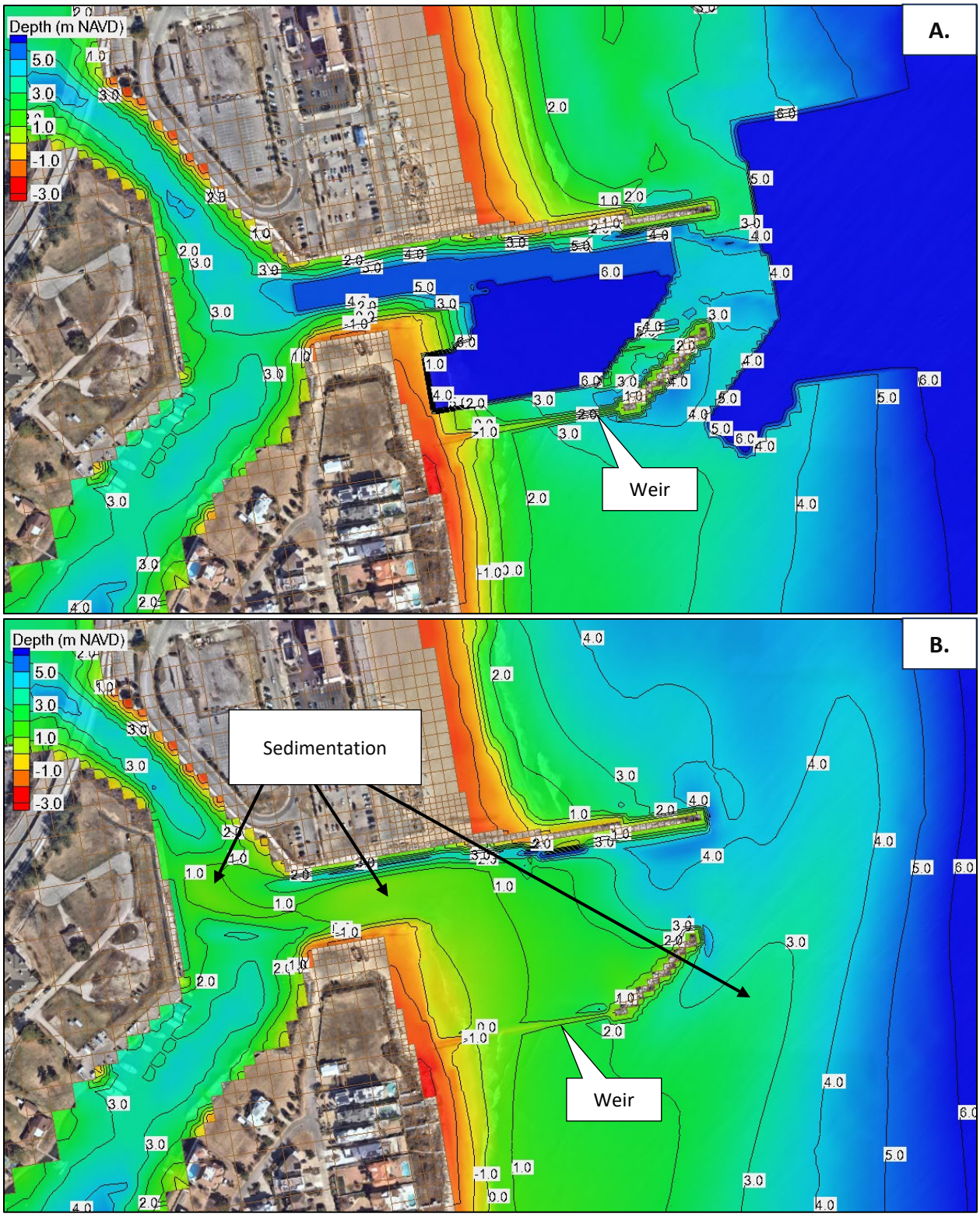


Figure 4-16: Initial and modeled elevations at Rudee Inlet after one year simulation during model calibration under AD initial bathymetry conditions*. A) initial elevation B) final modeled elevation (both in meters NAVD). Depths are positive.*

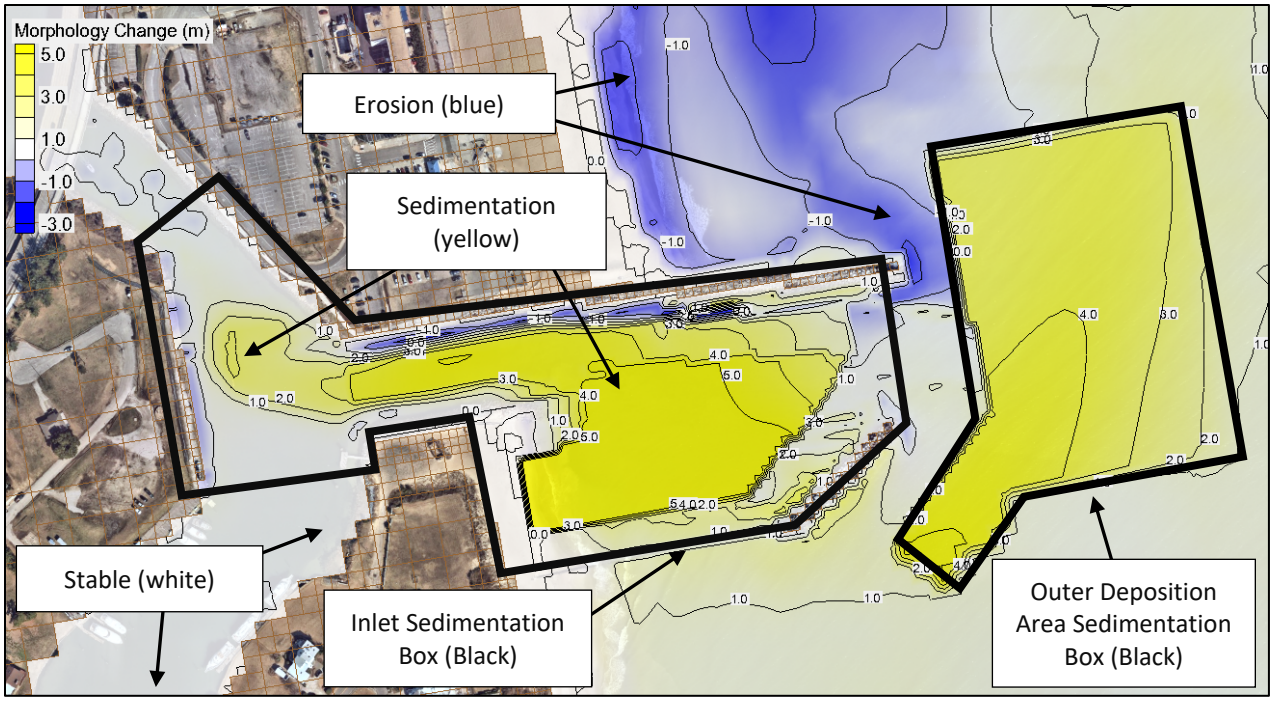
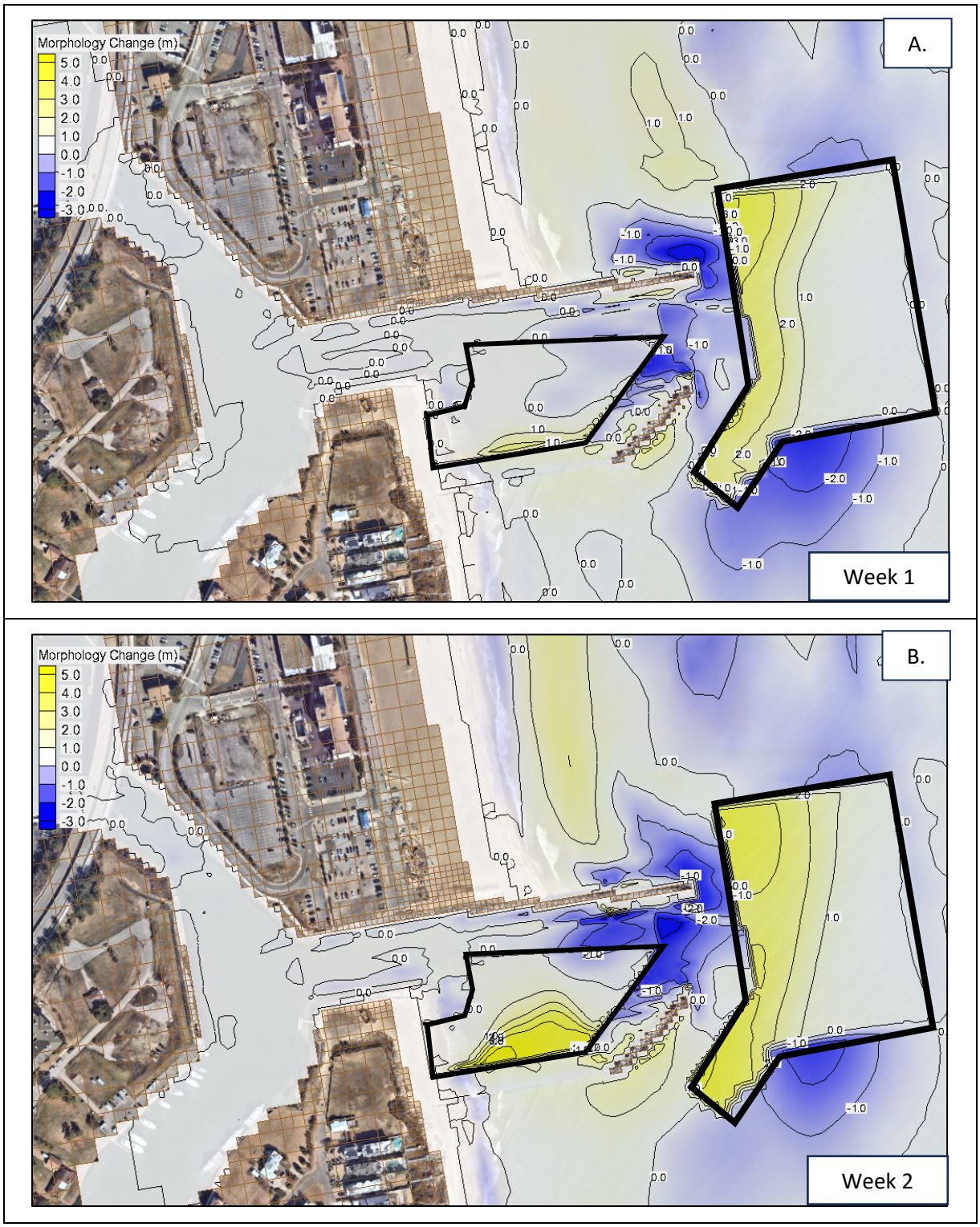
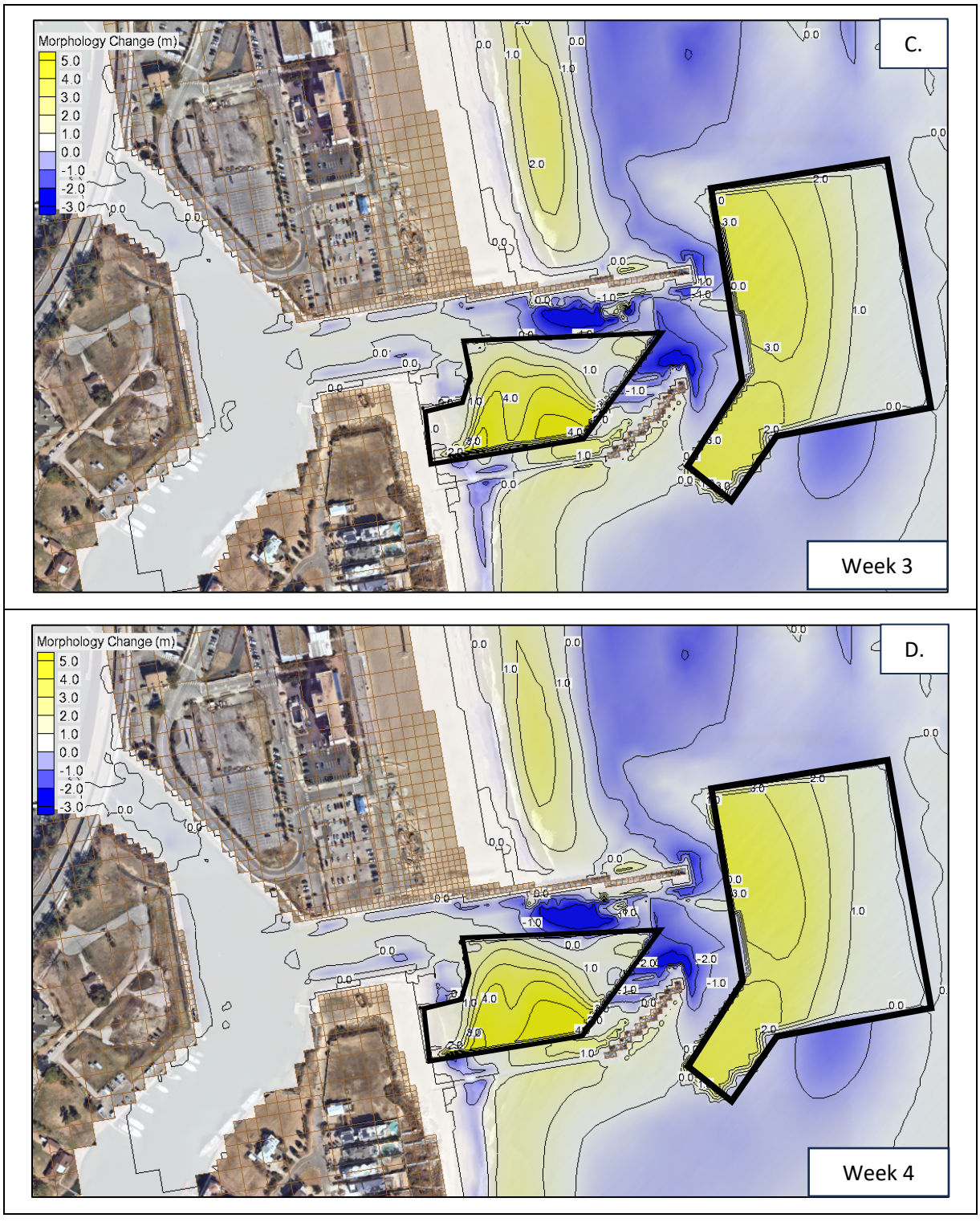


Figure 4-17: Sedimentation and erosion at Rudee Inlet after one year simulation during model calibration under AD initial bathymetry conditions. Areas of sedimentation are shown in yellow, and areas of erosion are shown in blue. Areas of negligible change are shown in white. The solid black lines show the areas within which inlet sedimentation was calculated.

Additionally, the sedimentation progression from the beginning of the simulation up to twelve weeks was assessed under AD initial bathymetry conditions with sediment transport increased. This scenario was chosen to assess maximum sedimentation rates immediately following a dredge event. Figure 4-18, A-F shows the sediment accumulation progression in 1-week intervals, and Table 4-4 shows the amount of sediment accumulated relative to each time period. Results reveal variability in sedimentation rate by week, due to the variability in wave and water level conditions by week. For example, the sedimentation rate is higher in the first few weeks post-dredge and then slows down. Another observation is that around week 10, a large wave event (wave height >3m/10ft) occurs and pushes the sand that had been accumulating in the outer deposition area into the inlet system, resulting in the large increase in sedimentation in the sand trap at the same time. This reveals that while most sediment buildup is a result of sediment transport over the weir, large wave events may also bring in additional sand through the inlet entrance. Results reveal more sediment accumulates in the inlet system under AD initial bathymetry conditions in comparison to PMI survey conditions because more sand settles in the dredged holes instead of traveling elsewhere in the model domain (i.e. offshore or further north towards the Resort Beach). Generally speaking, deeper dredge cuts promote increased sedimentation rates immediately following a dredge event. Dredged cuts cause reductions in currents/flows by inducing spreading over the water column, and a decrease in wave orbital velocities along the deepened sea bottom. These effects result in reduced bottom shear stress which promotes increased sedimentation in dredged areas.


Morphology Change (m)
5.0
4.0
3.0
2.0
1.0
0.0
-1.0
-2.0
-3.0
A.
Week 1
Morphology Change (m)
5.0
4.0
3.0
2.0
1.0
0.0
-1.0
-2.0
-3.0
B.
Week 2


Morphology Change (m)
5.0
4.0
3.0
2.0
1.0
0.0
-1.0
-2.0
-3.0
C.
Week 3
Morphology Change (m)
5.0
4.0
3.0
2.0
1.0
0.0
-1.0
-2.0
-3.0
D.
Week 4

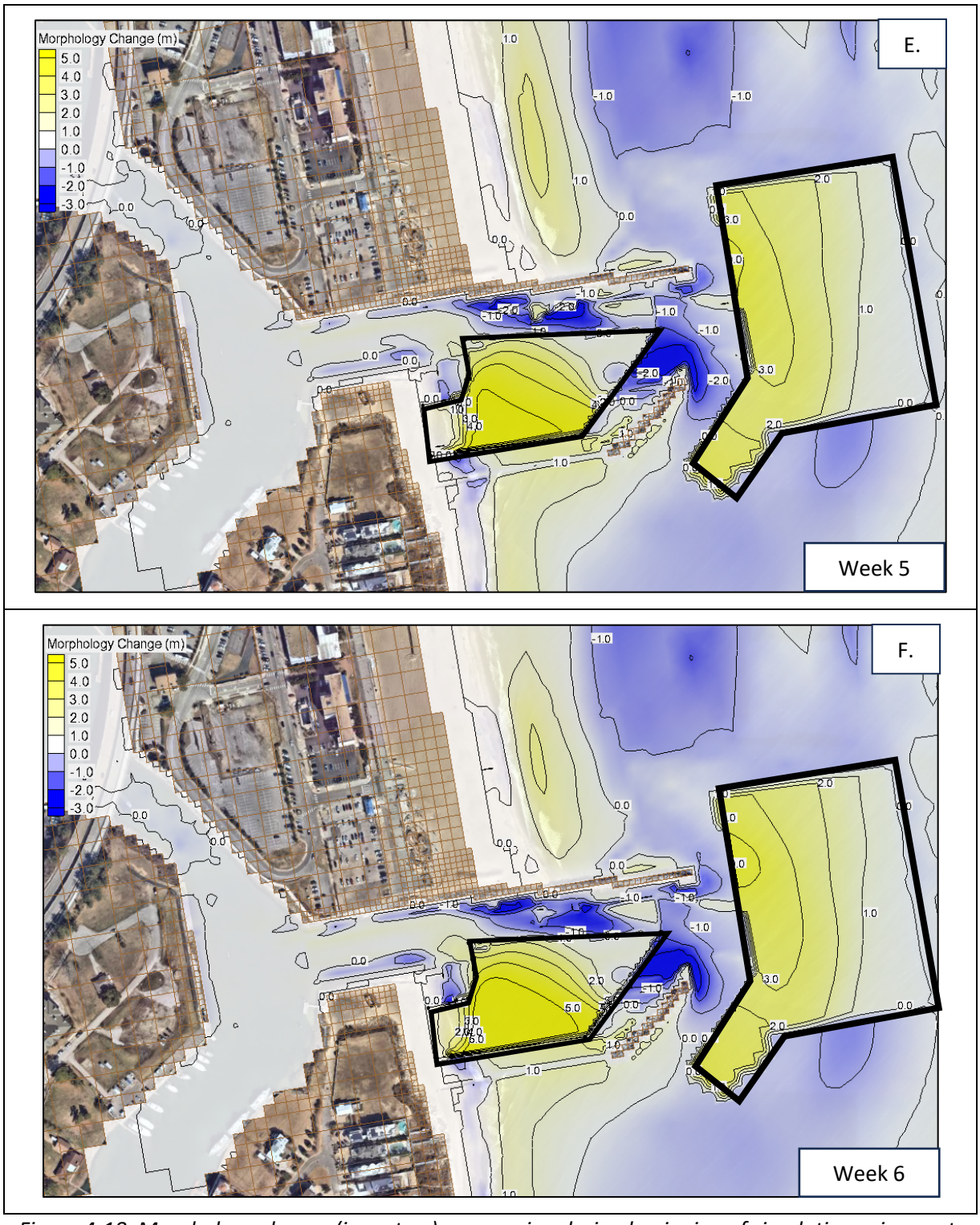


Figure 4-18. Morphology change (in meters) progression during beginning of simulation using post-dredge initial conditions for baseline model run. A) 1-week change; B) 2-week change; C) 3-week change; D) 4-week change; E) 5-week change; F) 6-week change. Yellow indicates accretion, blue indicates erosion, and white indicates negligible change. The dredged sand trap and dredged outer deposition basin are outlined with black solid lines.

Table 4-3: Sediment accumulation by week in sand trap and outer deposition areas under AD initial bathymetry dredge conditions with increased sediment transport. The first twelve weeks of the simulation were assessed.

Week	*Sand Trap Total Sediment Accumulation (cy)*	*Change in Sediment Accumulation by Week (cy)*	*Outer Deposition Basin Total Sediment Accumulation (cy)*	*Outer Deposition Basin Change in Sediment Accumulation by Week (cy)*
1	4,170	+4,170	52,670	+52,670
2	22,520	+18,360	71,540	+18,870
3	39,970	+17,440	94,500	+22,950
4	44,570	+4,600	95,900	+1,410
5	51,490	+6,920	92,160	-3,740
6	57,420	+5,930	96,300	+4,140
7	66,960	+9,540	99,740	+3,440
8	68,480	+1,520	100,460	+720
9	69,890	+1,410	113,150	+12,700
10	96,830	+26,940	38,020	-75,130
11	92,120	-4,710	50,090	+12,060
12	88,400	-3,730	56,100	+6,010

Model runs for Baseline Conditions and Proposed Alternatives in the remainder of this report represent PMI survey initial bathymetry conditions under which no dredging takes place before or during the year simulation. In reality, dredging occurs regularly in the inlet channel and sand trap area, preventing significant sediment buildup in these areas over time, especially in the western reaches of the inlet throat. Overall, the model does well in replicating the sedimentation and erosion processes in the study area. More detailed discussion, findings and figures are presented in the next section.

4.4 Model Results

4.4.1 Baseline Conditions Results

The Baseline Conditions model in this study represents conditions at Rudee Inlet with the existing weir using PMI survey initial bathymetry conditions. Results for the proposed groin extension alternatives were compared relative to results of the Baseline Conditions model. Figure 4-19 presents the sedimentation and erosion results for the Baseline Conditions model.

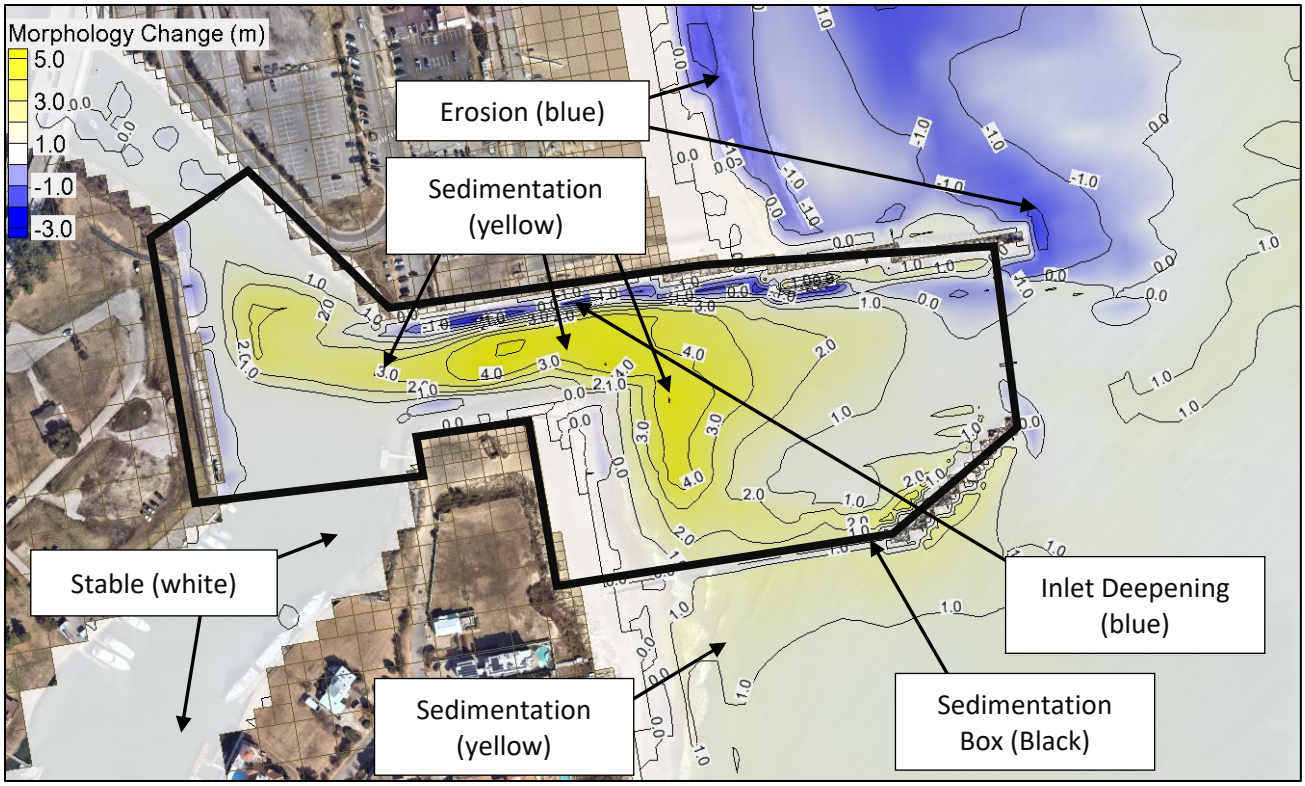


Figure 4-19: Sedimentation and erosion at Rudee Inlet after one year simulation under Baseline Conditions. Areas of sedimentation are shown in yellow, and areas of erosion are shown in blue. Areas of negligible change are shown in white. The solid black line shows the area within which sedimentation was calculated.

As presented in Figure 4-19, sedimentation within the sand trap and channel are evident after 1 year. Under Baseline Conditions, the inlet system accumulates ~97,850 cy of sediment over the one-year simulation. This is in range of recorded sediment quantities dredged and bypassed from the inlet on a yearly basis. The model does create a deeper inlet channel along the north jetty, which is shown for all model runs (under Baseline and Proposed Conditions). In addition to sedimentation spreading into the channel past the sand trap, waves and currents from the open ocean as well as the weir focus and reflect off the north jetty, creating this deeper area (noted as "Inlet Deepening" in Figure 4-19). This feature occurs in all model runs and the modeling analysis generally relies on relative differences between model runs and not absolute changes within each model run.

When examining the change in beach width and shoreline accretion along Croatan Beach downdrift of the weir (along the sand trap area shoreline), results reveal there is an increase in beach width and sediment accretion modeled under Baseline Conditions within the immediate vicinity of the inlet (Figure 4-20). While not unreasonable, in reality we may not expect this extent of gain due to regular dredging in the vicinity. Proposed conditions discussed in the next section will discuss changes to beach width on a relative basis to isolate change due to the groin extensions.



Figure 4-20: Modeled 0m NAVD (~MSL) contour location after one year simulation under Baseline Conditions.

Sediment transport vectors in the vicinity of the inlet system are shown in Figure 4-21. Transects were chosen at locations updrift/downdrift along the oceanfront and within the Rudee Inlet system. Transport vectors represent gross transport. The net sediment transport direction for each cross-shore and alongshore transect is shown with yellow arrows.

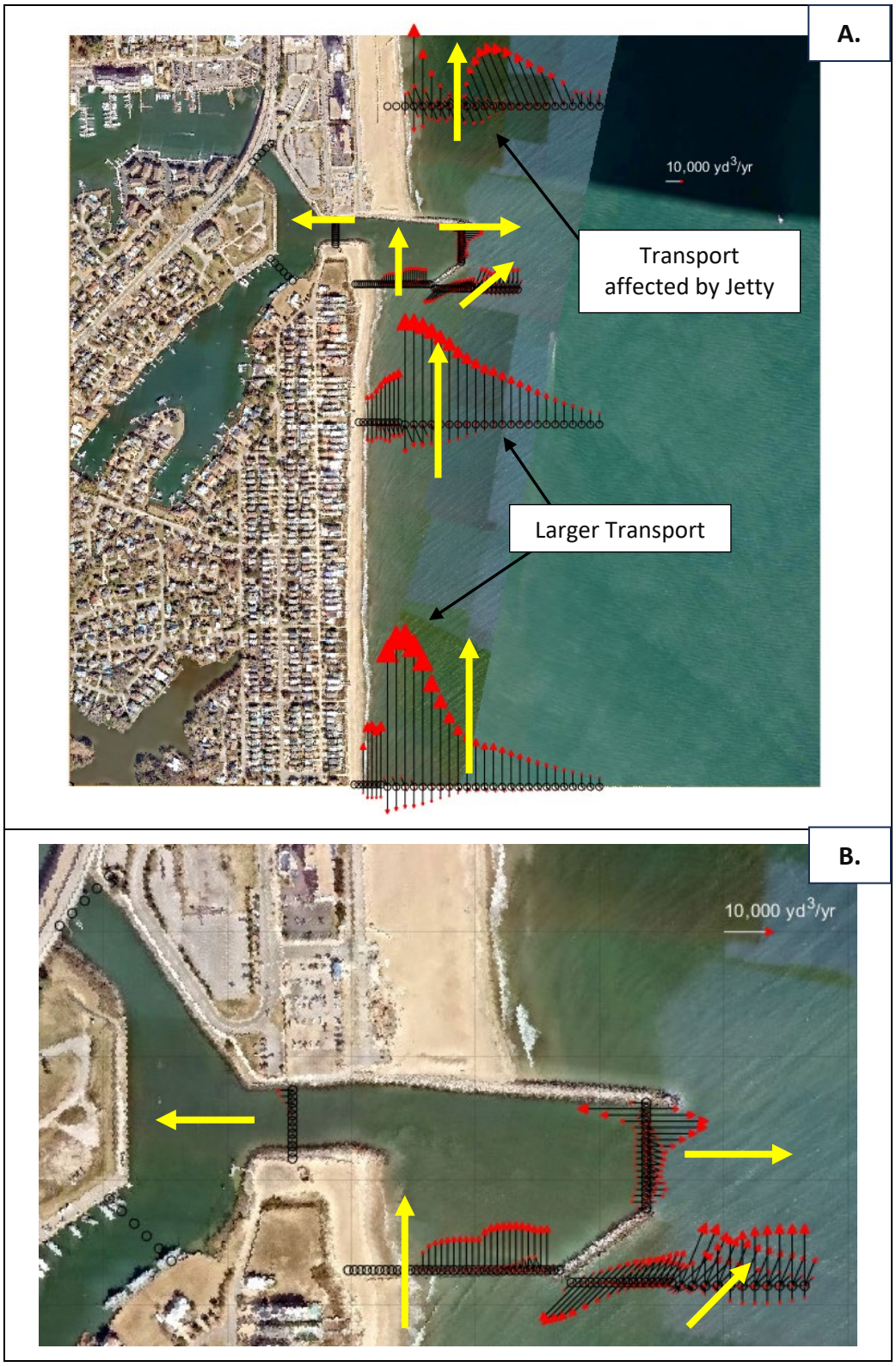


Figure 4-21: Gross sediment transport transects over 1-year simulation under Baseline Conditions. A) entire study area; B) zoom-in of Rudee Inlet. Overlaid with the net sediment transport direction for each transect (yellow arrows).

As expected, results show the dominant sediment transport direction as south to north, with the net direction of sediment transport directed through the weir and out of the inlet between the north jetty and breakwater.

For all the model runs, sediment transport vectors along the oceanfront shorelines are strongest in the nearshore (correlating with sandbars and surf zone). Sediment transport vectors in, out, and around the inlet generally replicate the known processes. Sediment transport vectors are binned in 22.5 degree increments (16 total). Oceanfront transport vectors are typically north/south bin dominated. For inlet system transect vectors, several different vector directions are noted and reflect the more complex processes at work within this system.

4.4.2 Proposed Conditions Results

A total of 9 Cases were tested in this numerical modeling assessment, listed below:

- **Case 1:** 20m (~65ft) effective groin extension only (90m/~295ft length weir remaining)
- **Case 2:** 30m (~100ft) effective groin extension only (80m/~260 length weir remaining)
- **Case 3:** 40m (~130ft) effective groin extension only (70m/~230ft length weir remaining)
- **Case 4:** 50m (~165ft) effective groin extension only (60m/~195ft length weir remaining)
- **Case 5:** 60m (~195ft) effective groin extension only (50m/~165ft length weir remaining)
- **Case 6:** 70m (~230ft) effective groin extension only (40m/~130ft length weir remaining)
- **Case 7:** 50m effective groin extension + 30m-long rock breakwater spur (60m/~195ft length weir remaining)
- **Case 8:** 50m (~165ft) effective groin extension + 35m-long rock south jetty extension (60m/~195ft length weir remaining)
- **Case 9:** 50m effective groin extension + raised weir elevation (+0.3m/1ft) (60m/~195ft length weir remaining)

In addition to raw model outputs, all model results were compared to a Baseline Case to assess relative change. For Cases 1-6, results were compared relative to Baseline Conditions to isolate the effect of the groin extension. For Cases 7-9, results were compared relative to Case 4 to isolate the effect of each added component in addition to the rock groin. As with most coastal process models, the CMS model is based on physics, and there can be areas of slight under or overprediction related to currents, wave dissipation, bottom friction responses, etc. By comparing model runs on a relative basis, these areas of slight under- or over-prediction get "zeroed out," which allows the reviewer to focus on the changes in physics between model runs. Intuitively, hydrodynamic and sedimentation processes farther away from the project should (and do) show relatively little or no differences (shown as white in the figures) between the proposed condition and baseline condition.

4.4.2.1 Cases 1-6: Groin Extension Results

The two primary goals of assessing the effective groin lengths were to 1) maintain or reduce sedimentation in the inlet system by reducing inflow over the weir, and 2) increase updrift recreational beach width at Croatan Beach. Table 4-4 presents the total quantity of sediment that accumulated in each scenario within the inlet system and the associated percent change in sedimentation relative to Baseline Conditions (see Figure 4-15 for boundary line within which quantities were computed). Relative percentage changes are useful for scaling any potential effects/impacts for more energetic years/conditions where transport rates are larger. Results reveal that as the effective groin length increases, the collective amount of sediment that accumulates in the inlet system decreases.

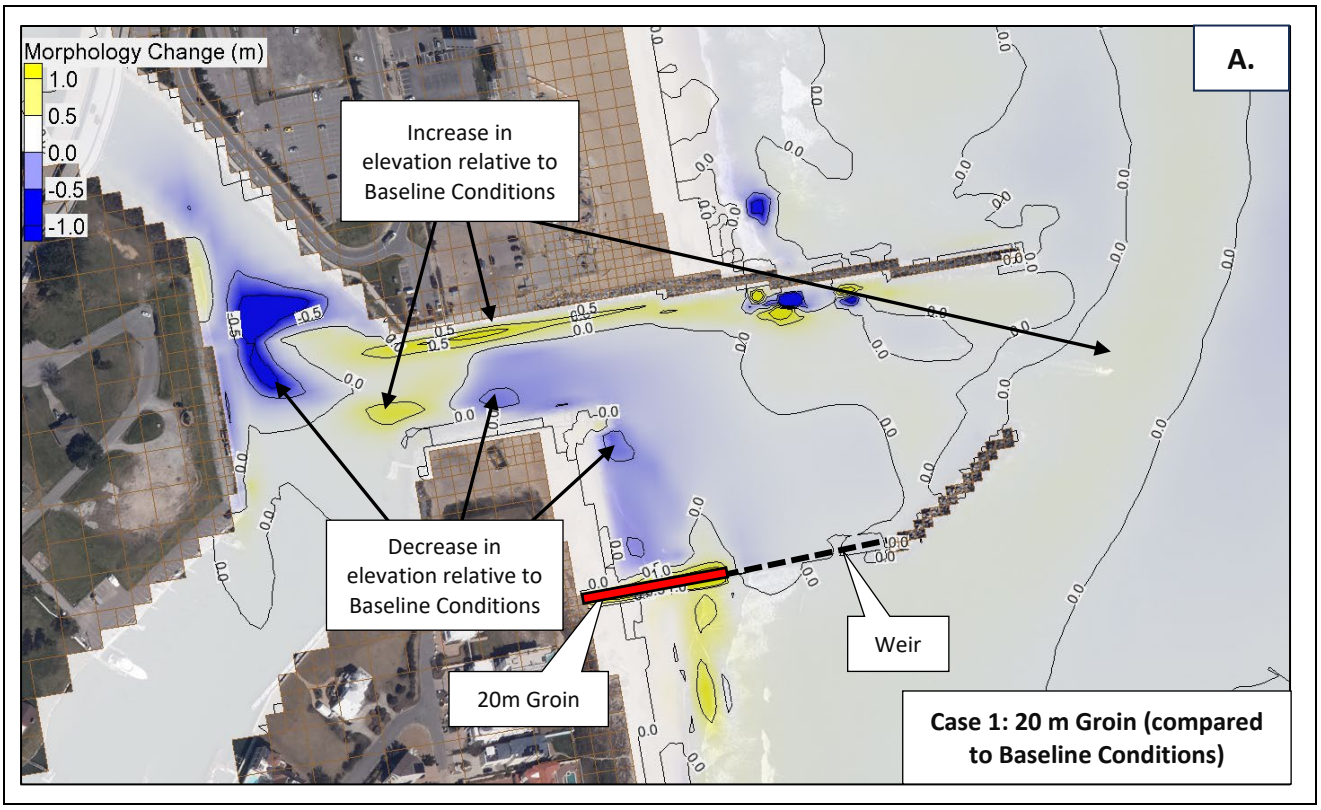
Table 4-4: Total quantity of sediment that accumulated within the inlet system in each scenario over a year simulation, and the percentage of change from Baseline Conditions.

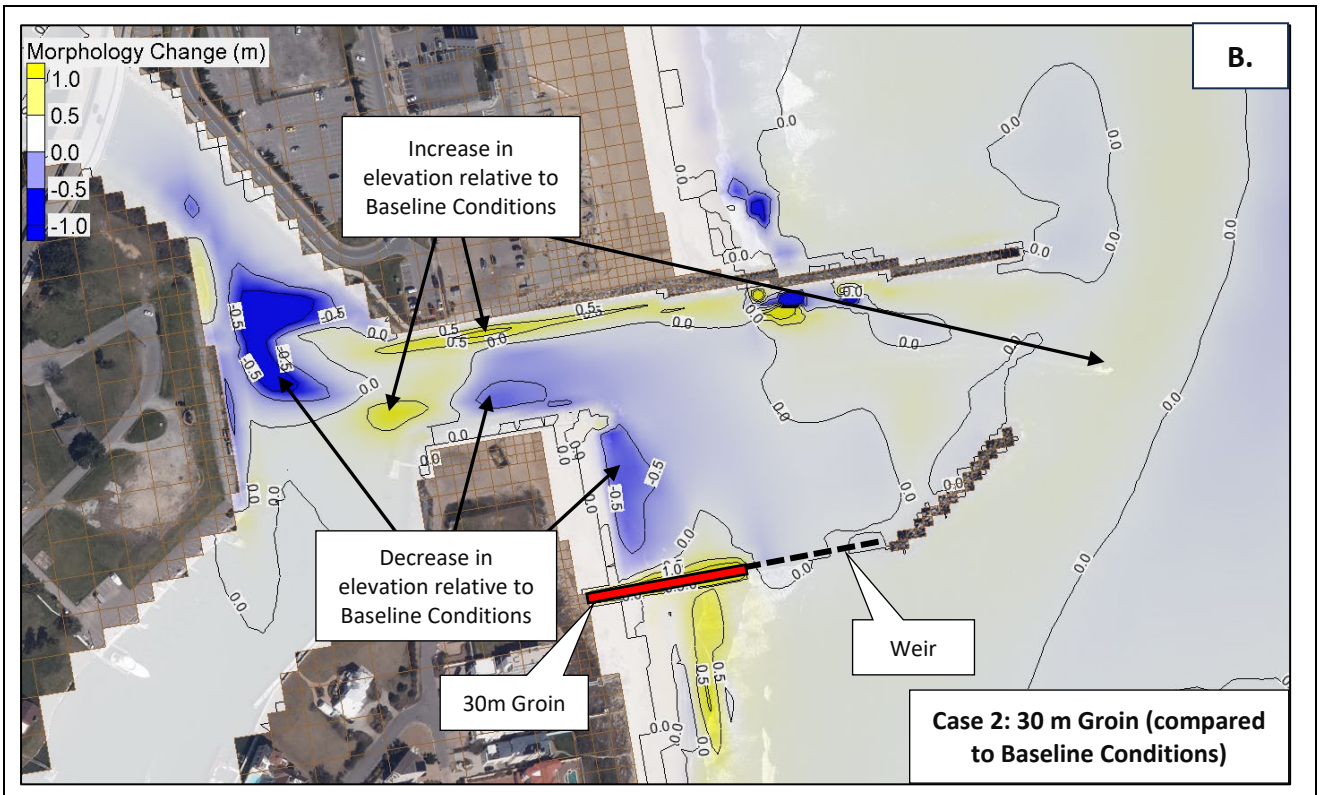
Scenario	Total Sedimentation in Inlet (cy)	Percent Change in Sedimentation from Baseline Conditions
Baseline Conditions	+97,850 cy	NA
Case 1: 20m groin	+93,270 cy	-5%
Case 2: 30m groin	+92,070 cy	-6%
Case 3: 40m groin	+90,840 cy	-7%
Case 4: 50m groin	+89,130 cy	-9%
Case 5: 60m groin	+87,760 cy	-10%
Case 6: 70m groin	+82,260 cy	-16%

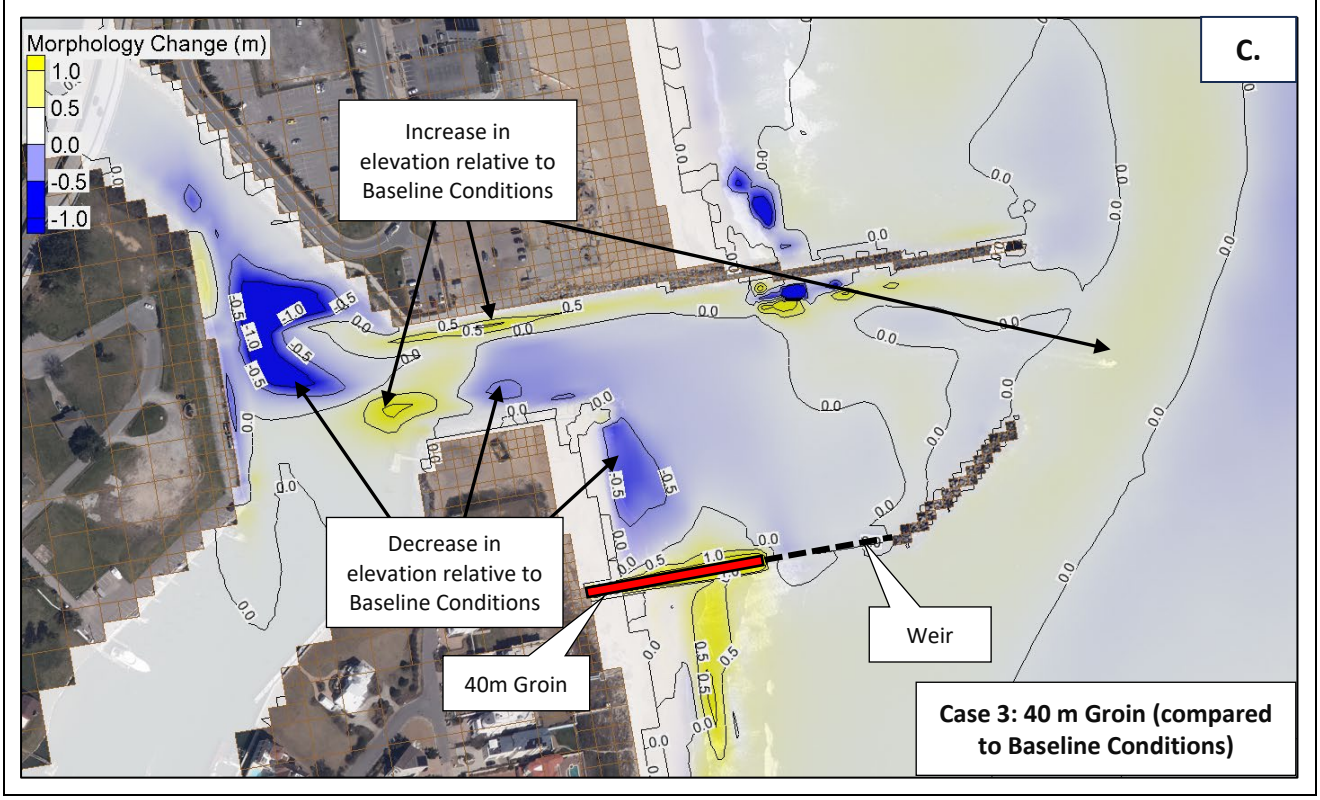
To better understand how patterns of sedimentation/erosion change due to each groin Case, the modeled elevations for each effective groin length were compared to the modeled elevations for Baseline Conditions, also referred to as morphology change (Figure 4-22 panels A-F). As shown in Figure 4-22, changes in morphology vary within the inlet system. Sedimentation is reduced in the sand trap area, parts of the inlet throat, and within the Lake Rudee entrance channel. In contrast, the northern side of the inlet throat and western edge of the south jetty saw increases in sedimentation relative to Baseline Conditions. However, at the northern side of the inlet throat immediately adjacent to the north jetty, part of the relative change is due to a decrease in erosion under the proposed groin scenario. In reviewing current modeling and wave modeling results for Cases 1-6, this is likely due to the change in currents entering and exiting the inlet (and some reflection off the north jetty). Overall, there is a net decrease in sediment accumulation in the inlet as the groin is lengthened (Table 4-4), and therefore a net benefit to adding a groin extension to the inlet system. Changes in morphology are relatively consistent between groin scenarios, but with higher degrees of change as the groin extension is lengthened.

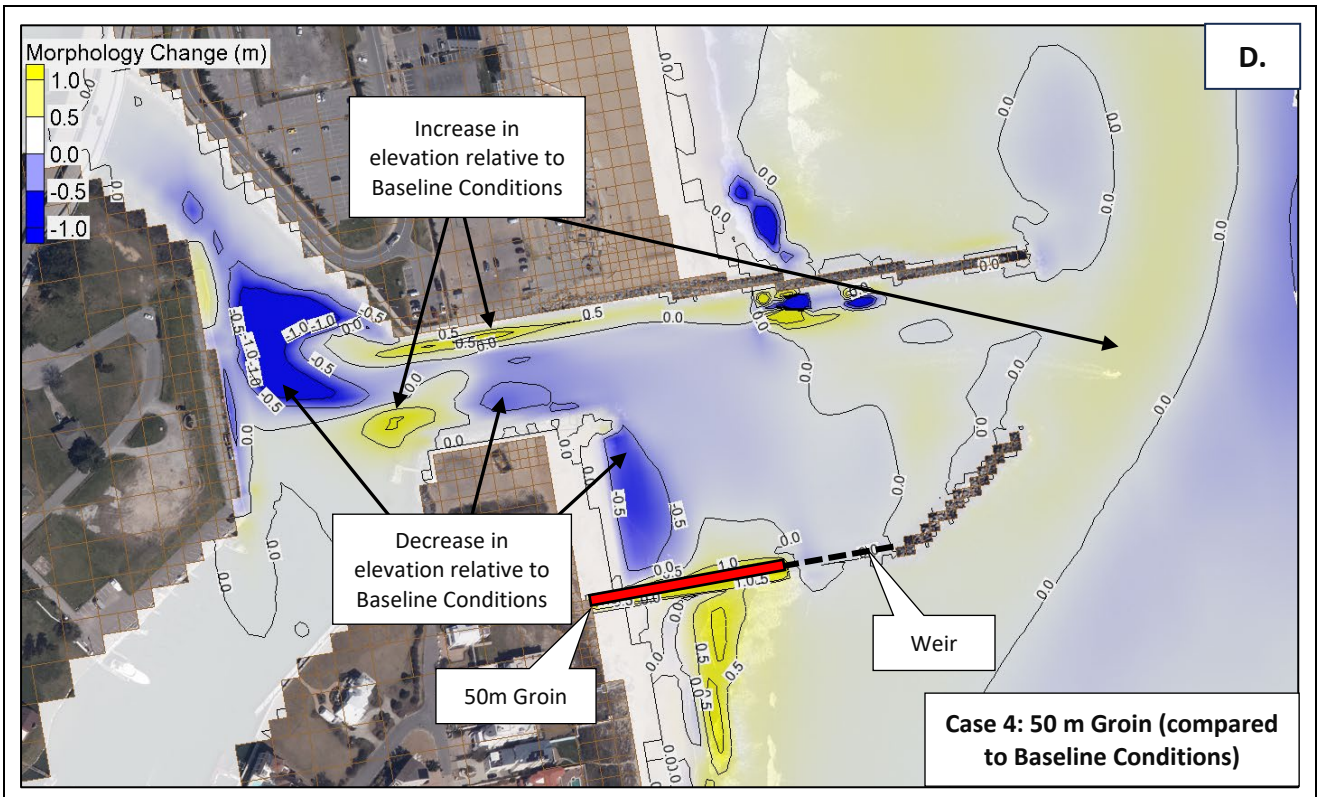
As the groin is lengthened, additional sediment collects on the updrift beach (i.e. Croatan Beach), as expected. However, sediment also collects along the outer reaches of the inlet and further north at the Resort Beach. Additional sedimentation at the inlet mouth due to a groin extension may present

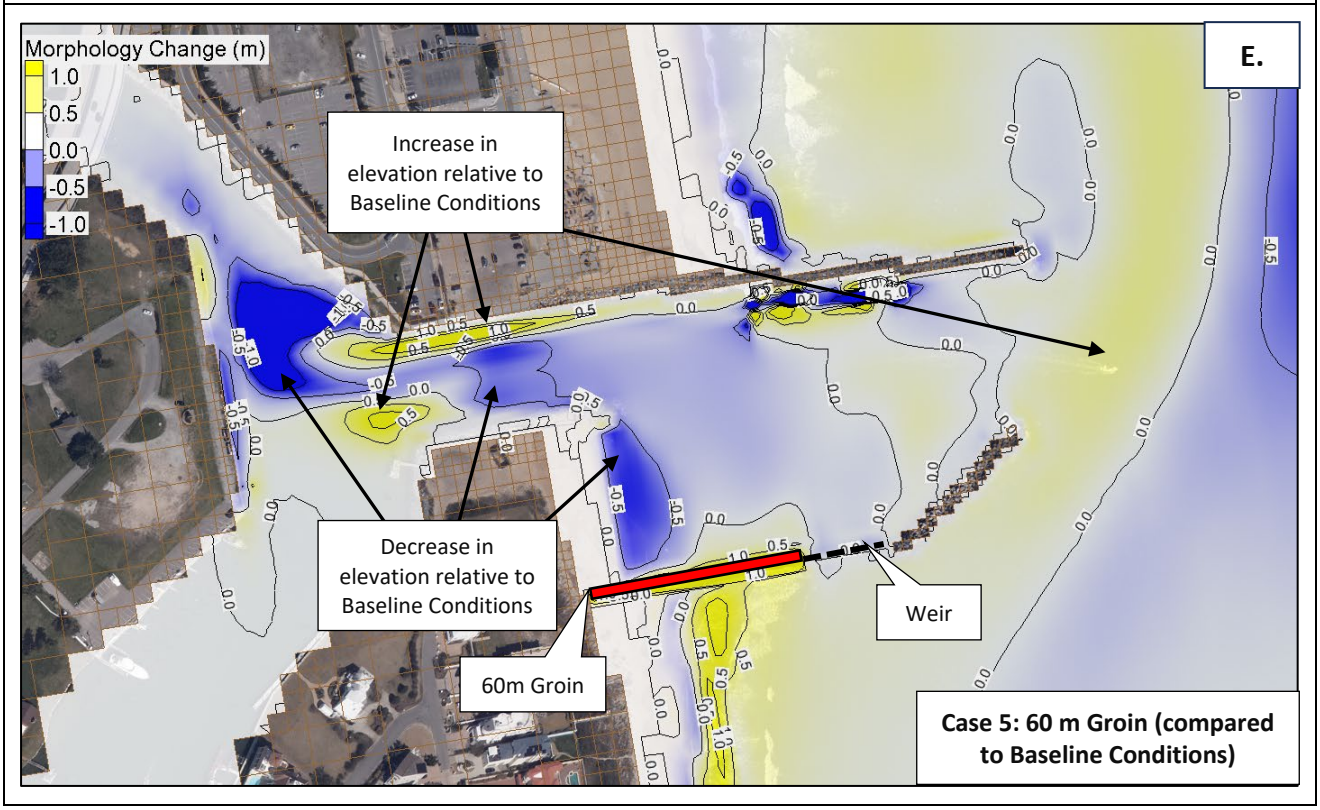
additional challenges, as sedimentation at the inlet mouth may exacerbate shoaling issues for vessel traffic. Therefore, selection of the preferred groin alternative for design should be considered a balance between the reduction of sediment transport through the weir, and an increase in sedimentation east of the inlet mouth due to changes in current patterns.




B.
Morphology Change (m)
1.0
0.5
0.0
-0.5
-1.0
Increase in elevation relative to Baseline Conditions
Decrease in elevation relative to Baseline Conditions
30m Groin
Weir
Case 2: 30 m Groin (compared to Baseline Conditions)


C.
Morphology Change (m)
1.0
0.5
0.0
-0.5
-1.0
Increase in elevation relative to Baseline Conditions
Decrease in elevation relative to Baseline Conditions
40m Groin
Weir
Case 3: 40 m Groin (compared to Baseline Conditions)


Morphology Change (m)
1.0
0.5
0.0
-0.5
-1.0
D.
Increase in elevation relative to Baseline Conditions
Decrease in elevation relative to Baseline Conditions
50m Groin
Weir
Case 4: 50 m Groin (compared to Baseline Conditions)


Morphology Change (m)
1.0
0.5
0.0
-0.5
-1.0
E.
Increase in elevation relative to Baseline Conditions
Decrease in elevation relative to Baseline Conditions
60m Groin
Weir
Case 5: 60 m Groin (compared to Baseline Conditions)

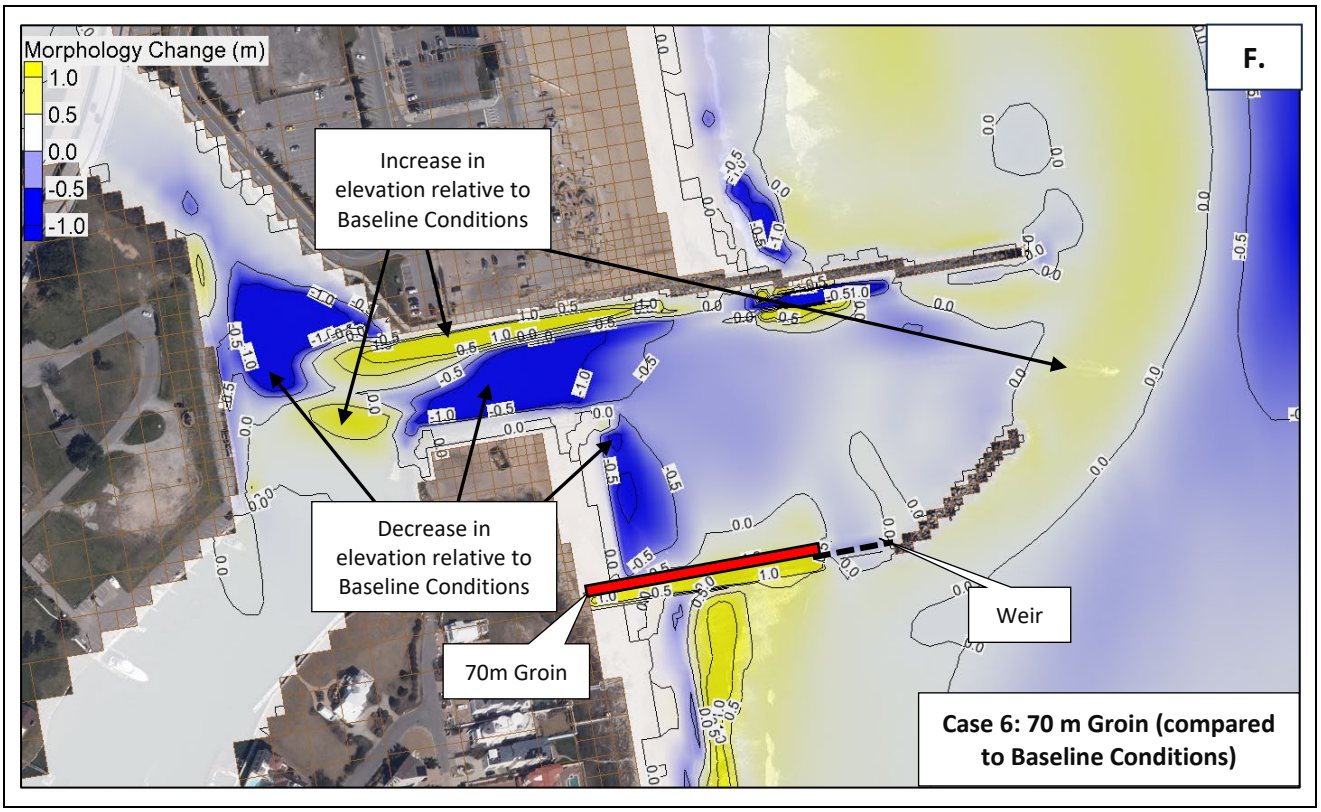


Figure 4-22: Modeled 1-year morphology change relative to Baseline Conditions scenario. A) Case 1; B) Case 2; C) Case 3; D) Case 4; E) Case 5; F) Case 6. Areas of more sedimentation or less erosion are shown in yellow, and areas of less sedimentation or more erosion are shown in blue. Areas of negligible change are shown in white.

The second primary goal of the study was to increase the beach width along Croatan Beach updrift of the weir. The 0m contour (~mean sea level) for each scenario was delineated to show increases to beach width updrift of each groin scenario, shown in Figure 4-23. The mean low water (MLW) line (-0.7m NAVD) was also delineated for each scenario (Figure 4-24).

The mean sea level (MSL) and mean low water lines were chosen for this analysis because the CMS model is generally better at modeling areas that are routinely wet/dry over the tidal cycle. Mean high water lines (and higher) are not as responsive to model changes as these areas are "dry" for the majority of model runs. Results reveal that as the groin length is increased, the cross-shore beach width increases, the MLW line moves seaward, and the alongshore impact distance of the groin increases. Table 4-5 shows the relative increase in updrift beach sediment quantities compared to Baseline Conditions (in cubic yards) and alongshore groin benefit distance.

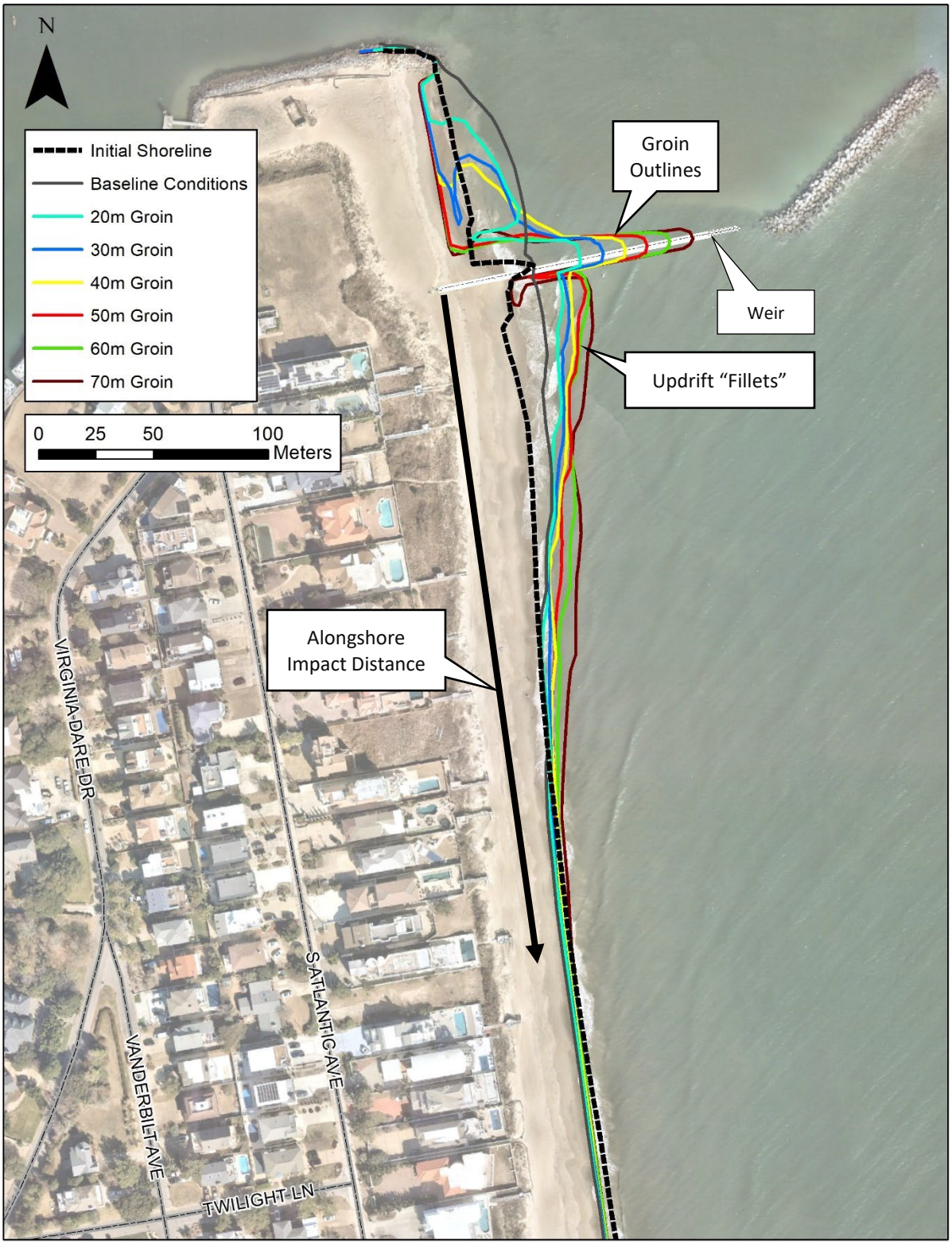


Figure 4-23: Modeled 0m NAVD contour (~mean sea level) location by scenario after one year simulation.



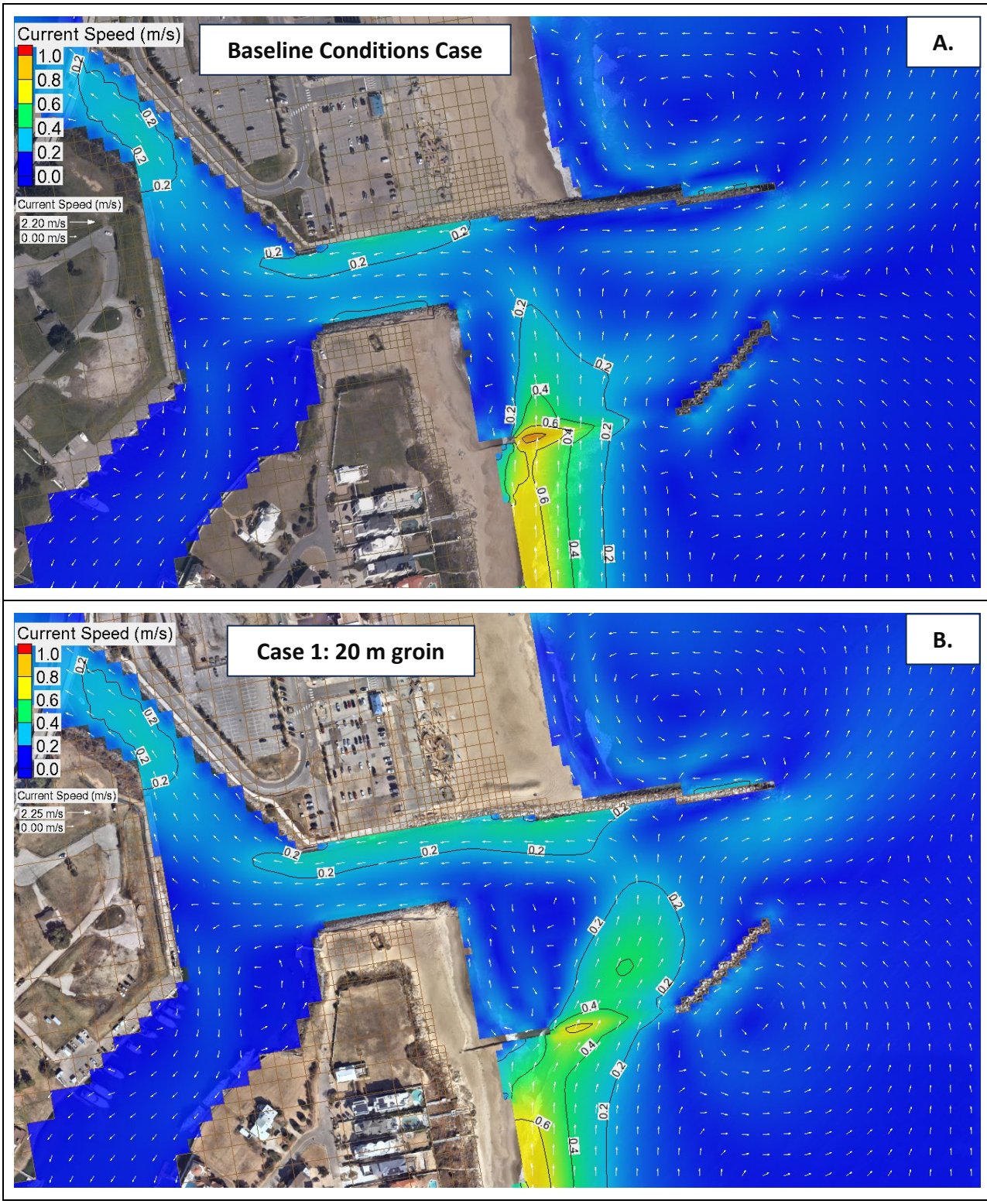
Figure 4-24: Modeled Mean Low Water (~0.7 NAVD) contour location by scenario after one year simulation.

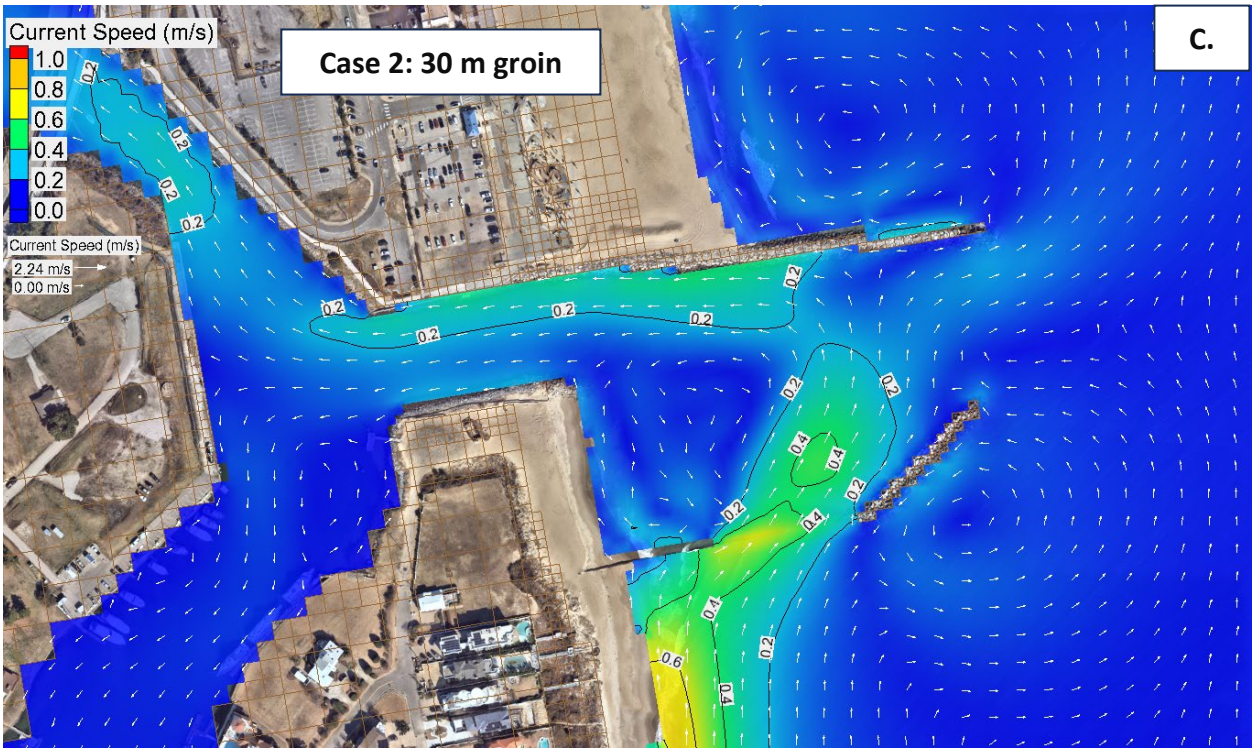
Table 4-5: Relative updrift beach sedimentation compared to Baseline Conditions and total alongshore sedimentation distance (i.e. alongshore groin impact distance).

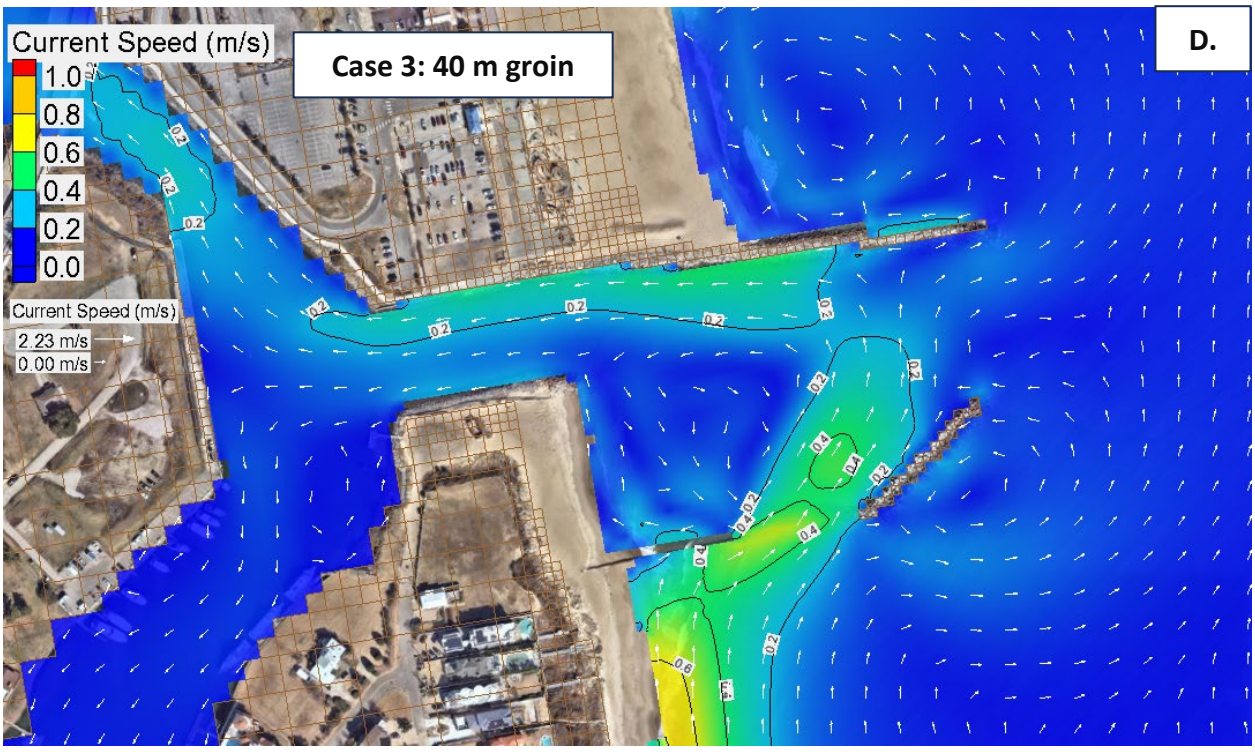
Scenario	Updrift Beach Sedimentation Relative to Baseline Conditions	Total Alongshore Sedimentation Distance using 0 contour (MSL)
Baseline Conditions	-	160 m (~525 ft)
Case 1: 20m groin	+4,190 cy	210 m (~690 ft)
Case 2: 30m groin	+5,890 cy	220 m (~720 ft)
Case 3: 40m groin	+8,140 cy	240 m (~790 ft)
Case 4: 50m groin	+8,350 cy	240 m (~790 ft)
Case 5: 60m groin	+11,410 cy	250 m (~820 ft)
Case 6: 70m groin	+12,210 cy	300 m (~980 ft)

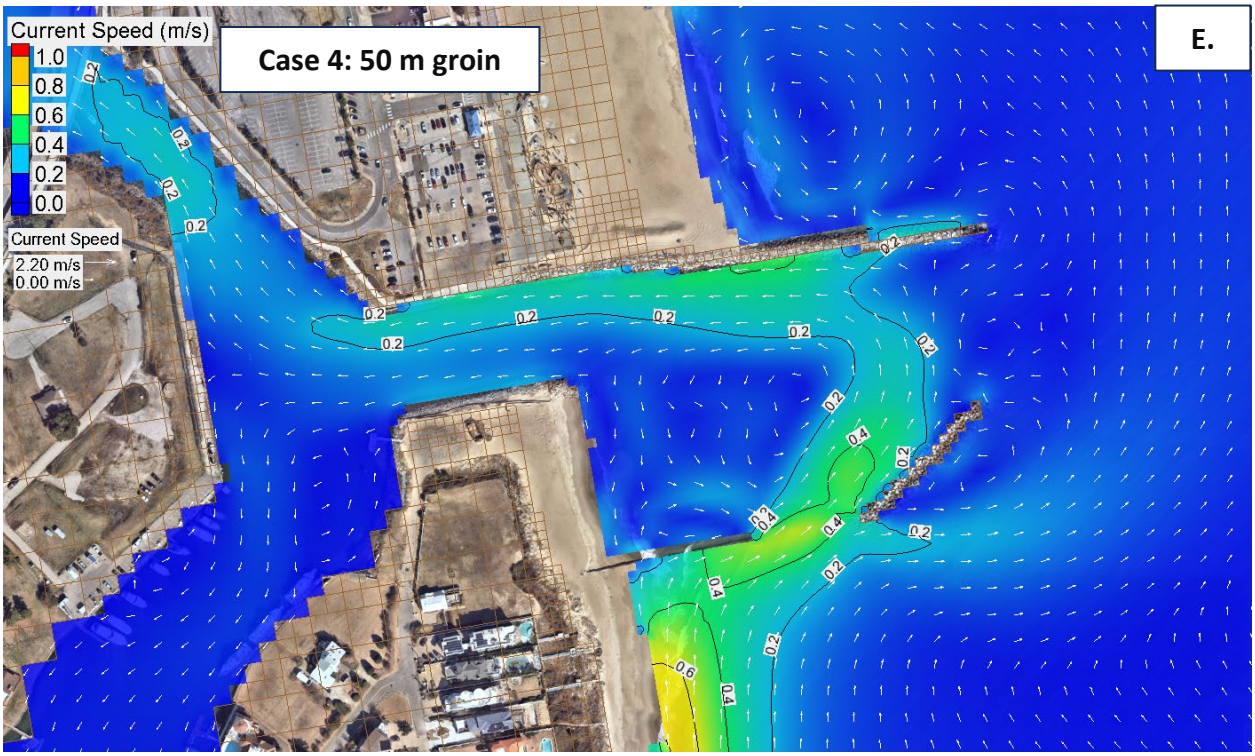
A potential concern is that constricting the flow over the weir may increase current speeds in the remaining portion of the weir. To understand how current speeds over the weir were affected by the groin, current speeds during a typical flood tide were compared between scenarios, shown in Figure 4-25, panels A-G (ebb tide current figures are shown later in the report). Results show that for a typical flood tide, current speeds over the weir show negligible change as the groin length is increased, with some reduction in current speeds in comparison to the Baseline Conditions Scenario.

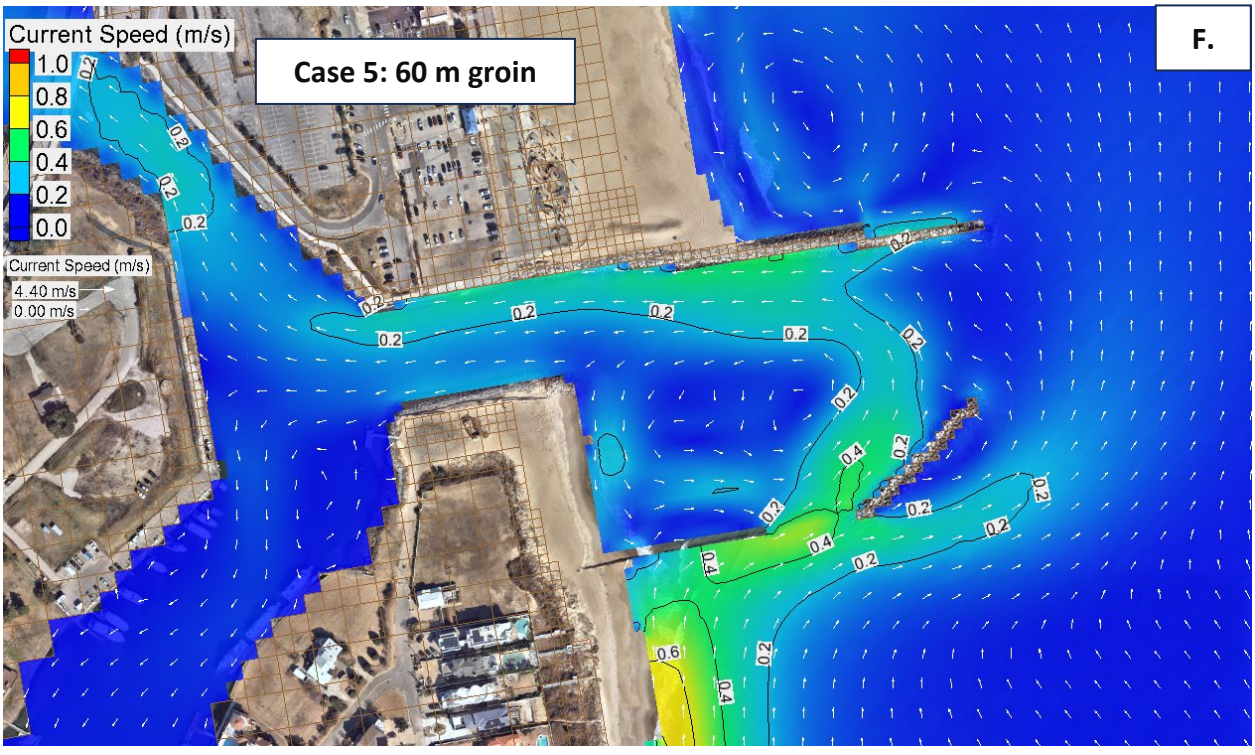
However, just downdrift of the weir, current speeds increase in the eastern portion of the inlet/sand trap area, and around the northern portion of the inlet throat as currents are redirected in the inlet due to groin extension. Results also reveal that as the groin length is increased, current speeds become increasingly directed to the northeast, away from the weir and around the breakwater and jetty system towards the Resort Beach. This effect is shown in the morphology figures where there is a relative increase in sedimentation near the inlet mouth and on the Resort Beach compared to Baseline Conditions.


Current Speed (m/s)
1.0
0.8
0.6
0.4
0.2
0.0
Current Speed (m/s)
2.20 m/s
0.00 m/s
Baseline Conditions Case
A.
Current Speed (m/s)
1.0
0.8
0.6
0.4
0.2
0.0
Current Speed (m/s)
2.25 m/s
0.00 m/s
Case 1: 20 m groin
B.


Current Speed (m/s)
1.0
0.8
0.6
0.4
0.2
0.0
Current Speed (m/s)
2.24 m/s
0.00 m/s
Case 2: 30 m groin
C.


Current Speed (m/s)
1.0
0.8
0.6
0.4
0.2
0.0
Current Speed (m/s)
2.23 m/s
0.00 m/s
Case 3: 40 m groin
D.


Current Speed (m/s)
1.0
0.8
0.6
0.4
0.2
0.0
Current Speed
2.20 m/s
0.00 m/s
Case 4: 50 m groin
E.


Current Speed (m/s)
1.0
0.8
0.6
0.4
0.2
0.0
Current Speed (m/s)
4.40 m/s
0.00 m/s
Case 5: 60 m groin
F.

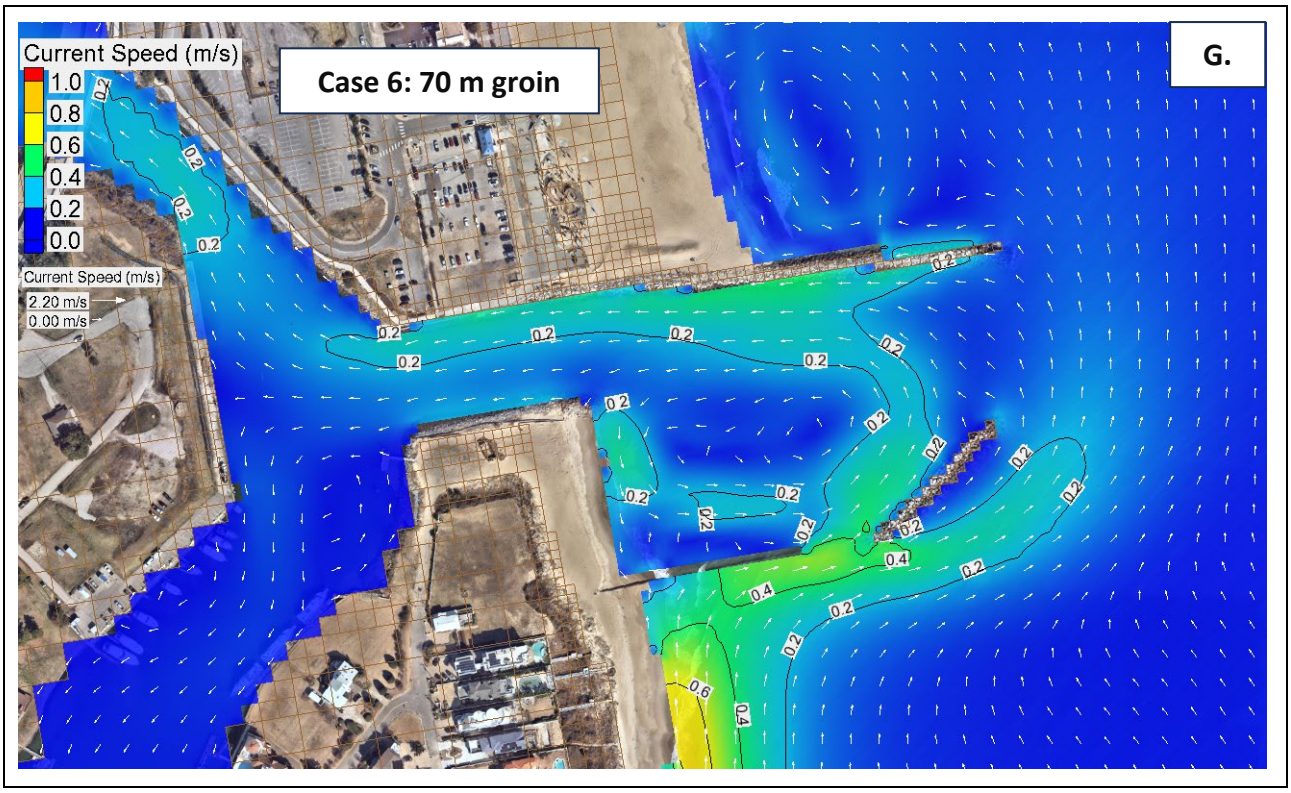
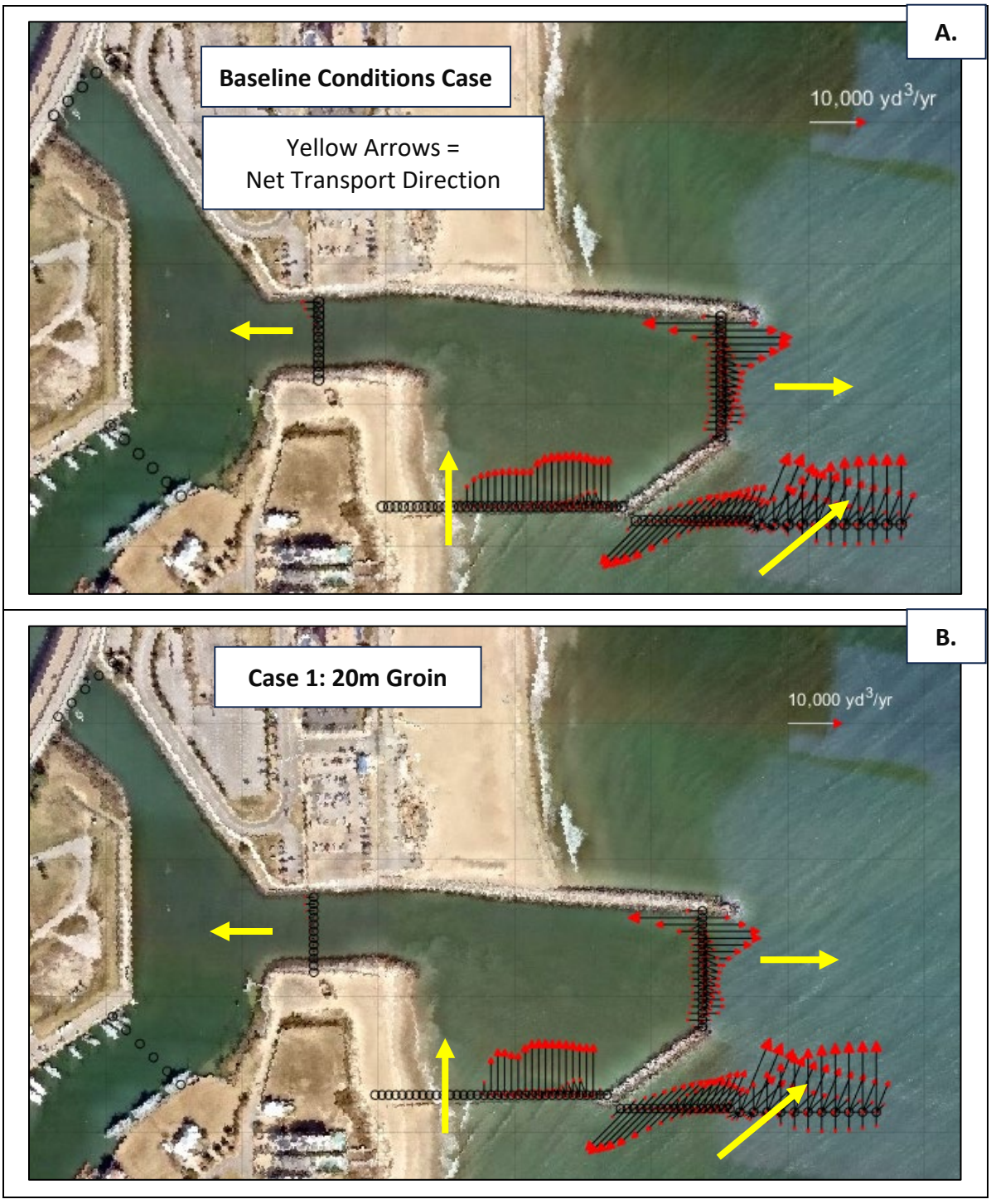
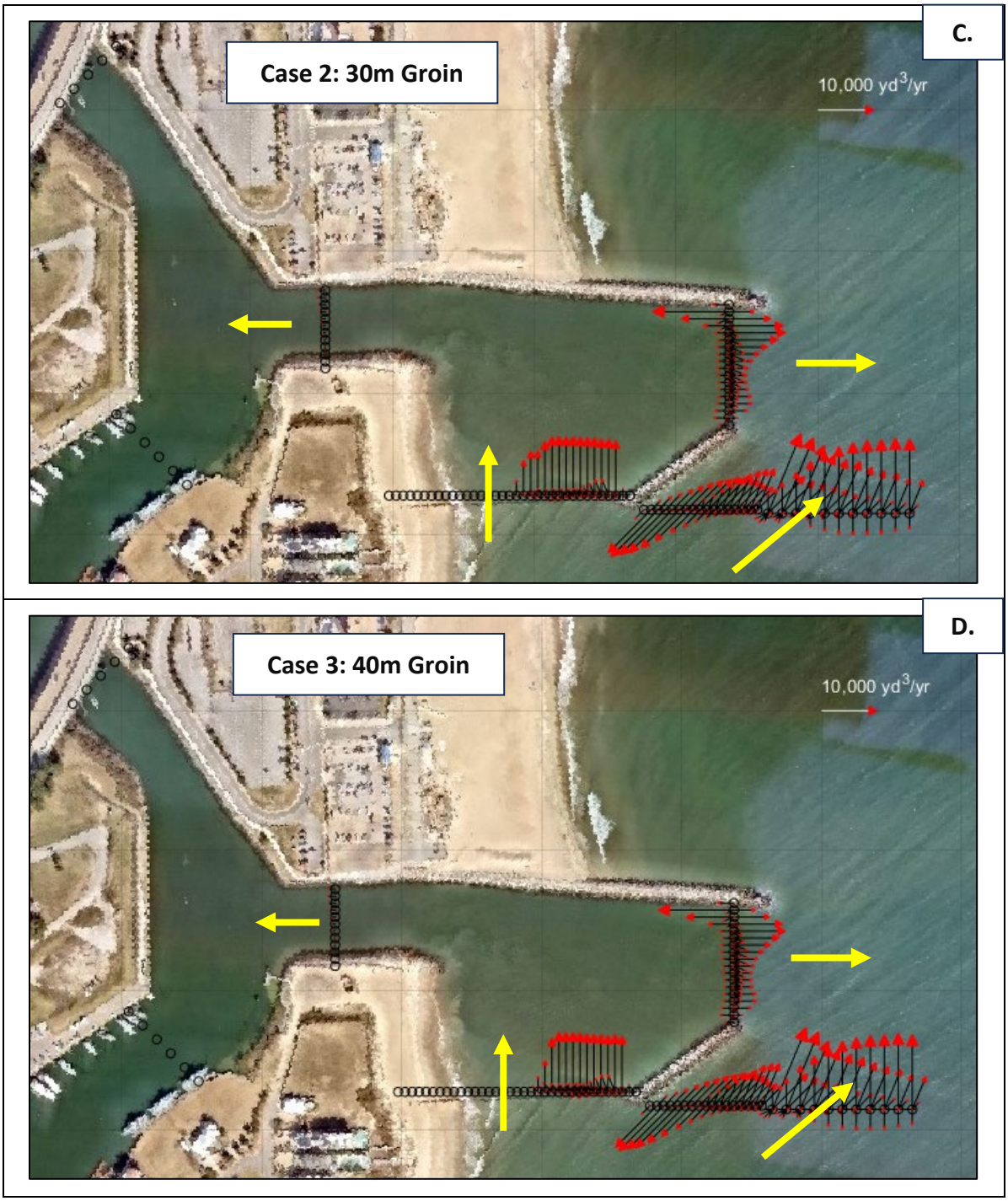


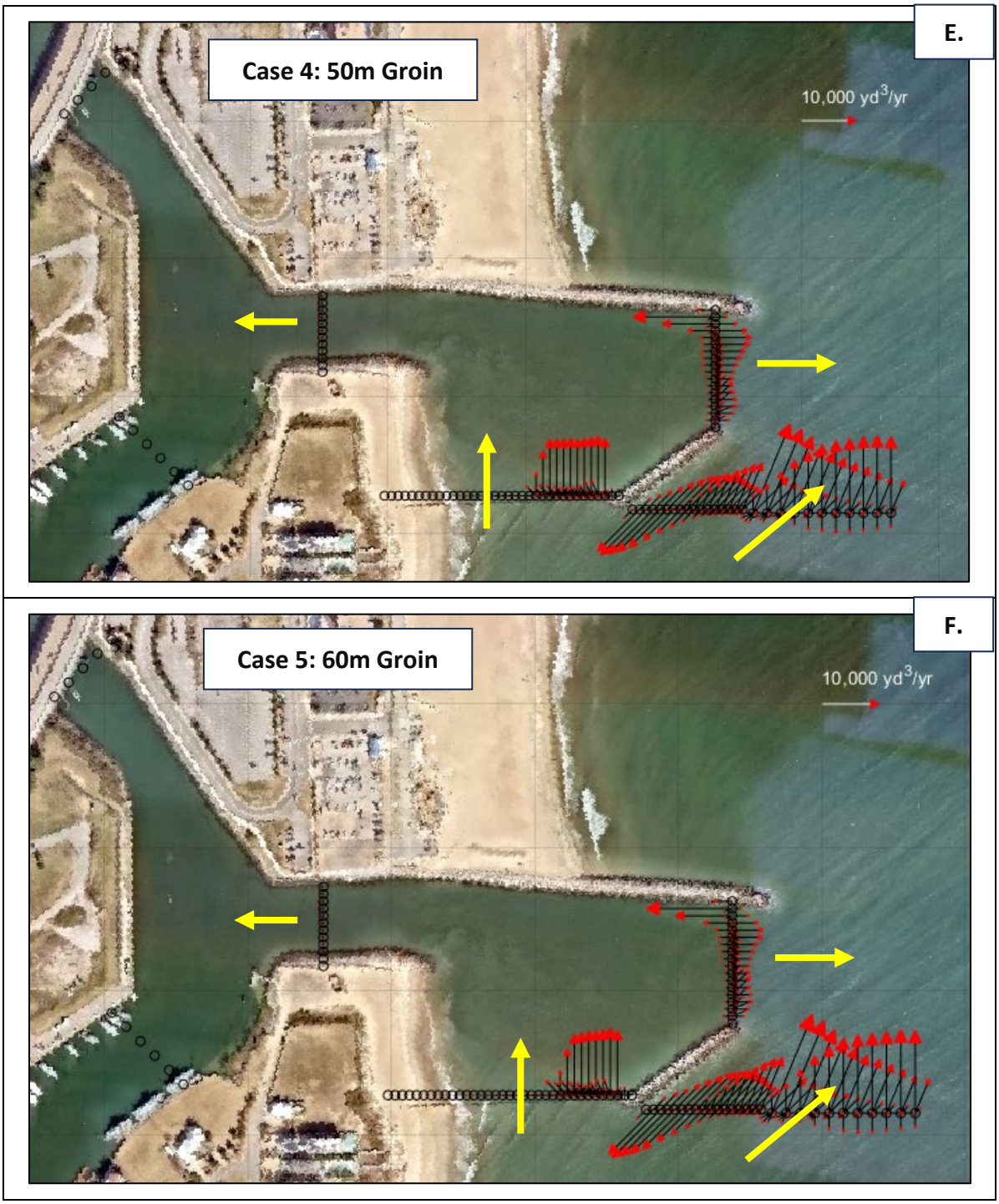
Figure 4-25: Modeled current speeds during a typical flood tide. A) Baseline Conditions; B) Case 1; C) Case 2; D) Case 3; E) Case 4; F) Case 5; G) Case 6.

Sediment transport magnitude and directionality around Rudee Inlet was also assessed for all scenarios, shown in Figure 4-26, panels A-G. The vectors represent the direction and scaled magnitude of sediment transport over simulations.

Results reveal that when flow is shifted over the weir due to the groin extensions, there is also a decrease in sediment leaving the inlet. This effect is also noted in the morphology change figures where relatively small decreases in sedimentation occur in the inlet as the groin is lengthened. An additional observation is that as the groin is lengthened the sediment transport seaward of the breakwater increases to the northeast, indicating that some of the sediment that would normally go into the inlet through the weir is being redirected around the breakwater instead (in addition to beneficial sediment accumulation on the updrift shoreline).


A.
Baseline Conditions Case
Yellow Arrows =
Net Transport Direction
10,000 yd3/yr
B.
Case 1: 20m Groin
10,000 yd3/yr


C.
Case 2: 30m Groin
10,000 yd3/yr
D.
Case 3: 40m Groin
10,000 yd3/yr


E.
Case 4: 50m Groin
10,000 yd3/yr
F.
Case 5: 60m Groin
10,000 yd3/yr

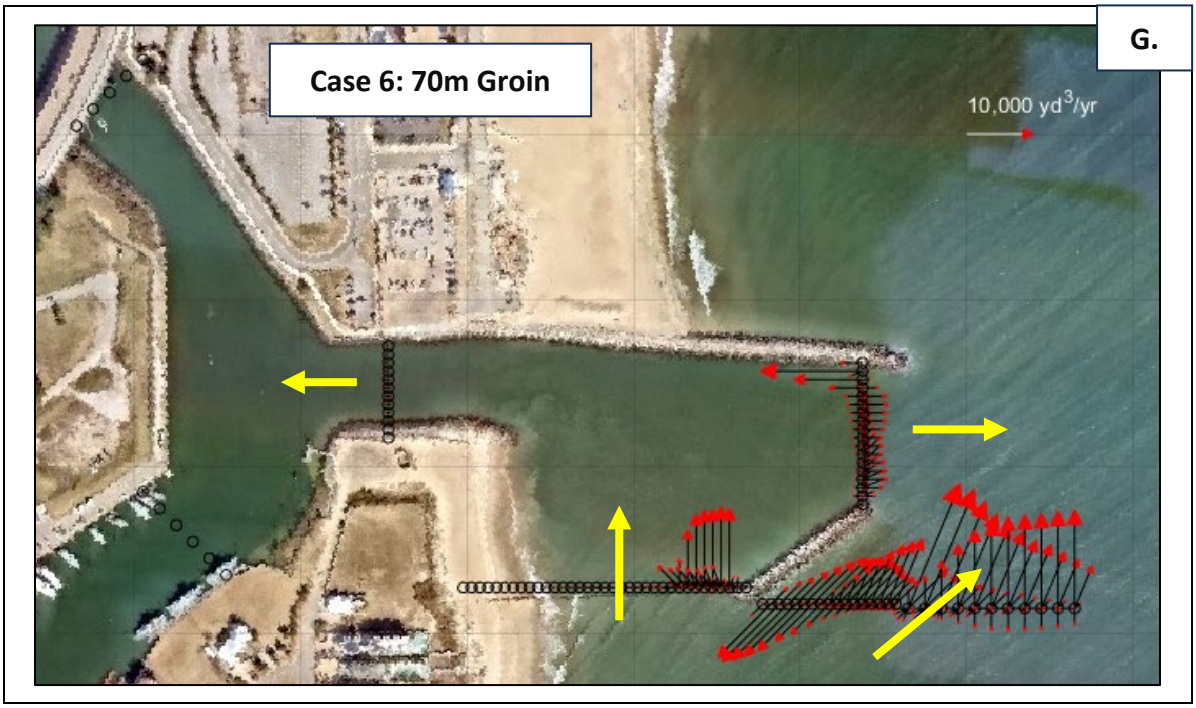


Figure 4-26: Gross sediment transport vector transects for Baseline Conditions and Cases 1-6. A) Baseline Conditions; B) Case 1; C) Case 2; D) Case 3; E) Case 4; F) Case 5; G) Case 6. Overlaid with the net sediment transport direction for each transect (yellow arrows).

4.4.2.2 Case 7: Breakwater Spur Results

In addition to the groin/weir length dimensioning Cases 1-6, the Case 7 simulation included the 50m groin length from Case 4 with an additional rock spur feature extending off the breakwater. The primary goal of Case 7 was to minimize sediment going offshore and accumulating at the inlet mouth and instead redirect more sediment into the inlet sand trap and updrift of the proposed extended rock groin.

When examining the collective amount of sediment that accumulated in the inlet system, results reveal the total amount of sediment decreased from 89,130 cy in Case 4 to 86,190 cy in Case 7 (a reduction of 2,940 cy, or 3%). This represents a minimal reduction in sedimentation in the inlet system over the course of a year, however, it is still a net benefit relative to Baseline Conditions which accumulated 97,850 cy over a year (total reduction of 11,660 cy, or 12%).

To better understand how patterns of sedimentation/erosion change due to the addition of the breakwater spur extension, the modeled elevations for Case 7 were compared to the modeled elevations for Case 4 (i.e., morphology change relative comparison). Using modeled elevations for Case 4 (50 m groin extension only) instead of the Baseline Conditions isolates the effect of the added rock extension. Figure 4-27 presents 1-year morphology change elevations for Case 7 relative to Case 4. As shown in Figure 4-27, changes in morphology vary within the inlet system. Positive values (yellow) indicate Case 7 sedimentation in comparison to Case 4. Negative values (blue) indicate Case 7 erosion in comparison to Case 4. Results reveal that with the addition of the rock spur extension (in addition to the 50m groin), there is more sedimentation to the west of the rock spur extension and updrift of the groin, as expected.

Other areas that show a relative increase in sedimentation relative to Case 4 are the southern section of the inlet throat, along the downdrift shoreline of the groin, and adjacent to the weir. This is likely due to sediment moving around the inlet due to the change in currents entering and exiting the inlet. Overall, there is a net decrease in sedimentation in the inlet with the breakwater spur added relative to both Case 4 and Baseline Conditions, and therefore a net benefit to adding the rock extension to the inlet system.

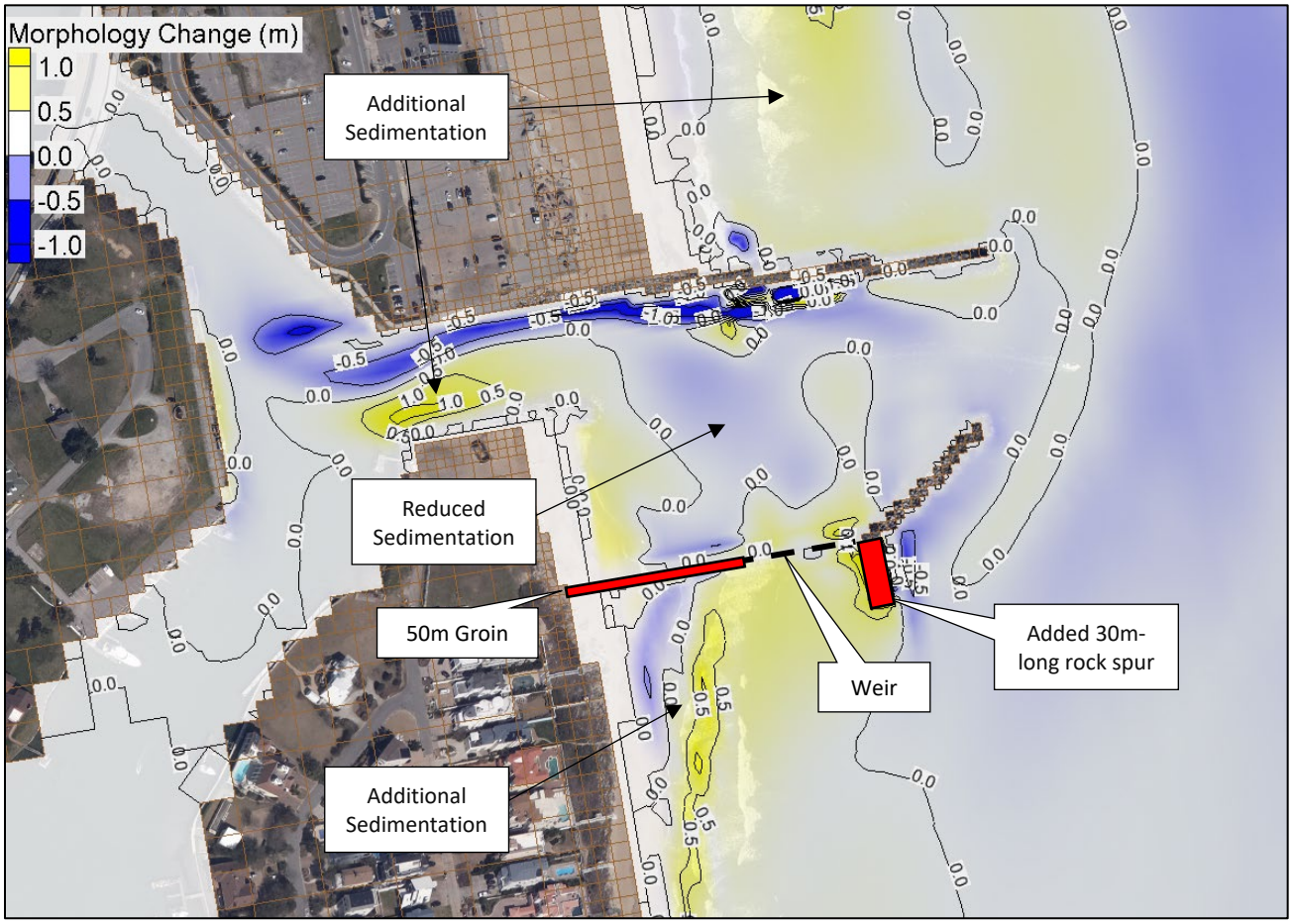


Figure 4-27: Morphology change due to the rock spur extension off the offshore breakwater relative to Case 4 (50m groin only). Areas of more sedimentation (or less erosion) are shown in yellow, and areas of more erosion (or less sedimentation) are shown in blue. Areas of negligible change are shown in white.

Sedimentation is reduced in parts of the sand trap area, the northern edge of the inlet throat, and the northwestern section of the inlet throat within the Lake Rudee entrance channel (Figure 4-27). There is some spatial variability in sedimentation patterns in areas offshore to the east and at the inlet mouth, but changes are negligible. The yellow areas north of the inlet at the Resort Beach are for the most part more indicative of less erosion in Case 7 relative to Case 4. Figure 4-28 shows the morphology change relative to the start of each respective run for reference. This indicates that sediment is being redirected from moving offshore or accumulating at the inlet mouth and is more directed through the weir and to the north past the inlet, effectively creating a more effective bypassing route to the Resort Beach. This could potentially allow more sand to bypass and accumulate on the Resort Beach.

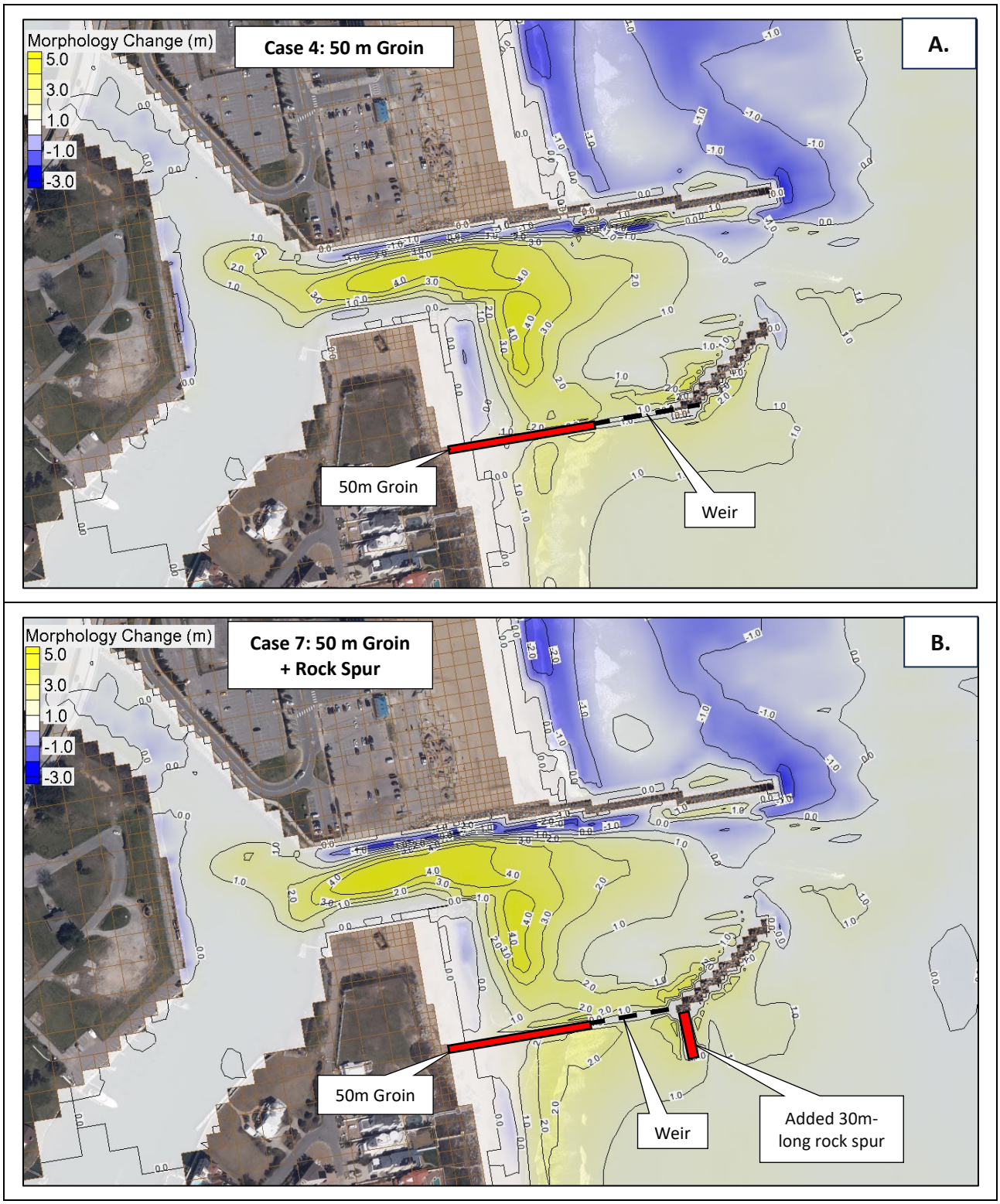


Figure 4-28: Morphology change relative to the start of the simulation for A) Case 4, and B) Case 7. Areas of sedimentation are shown in yellow, and areas of erosion are shown in blue. Areas of negligible change are shown in white.

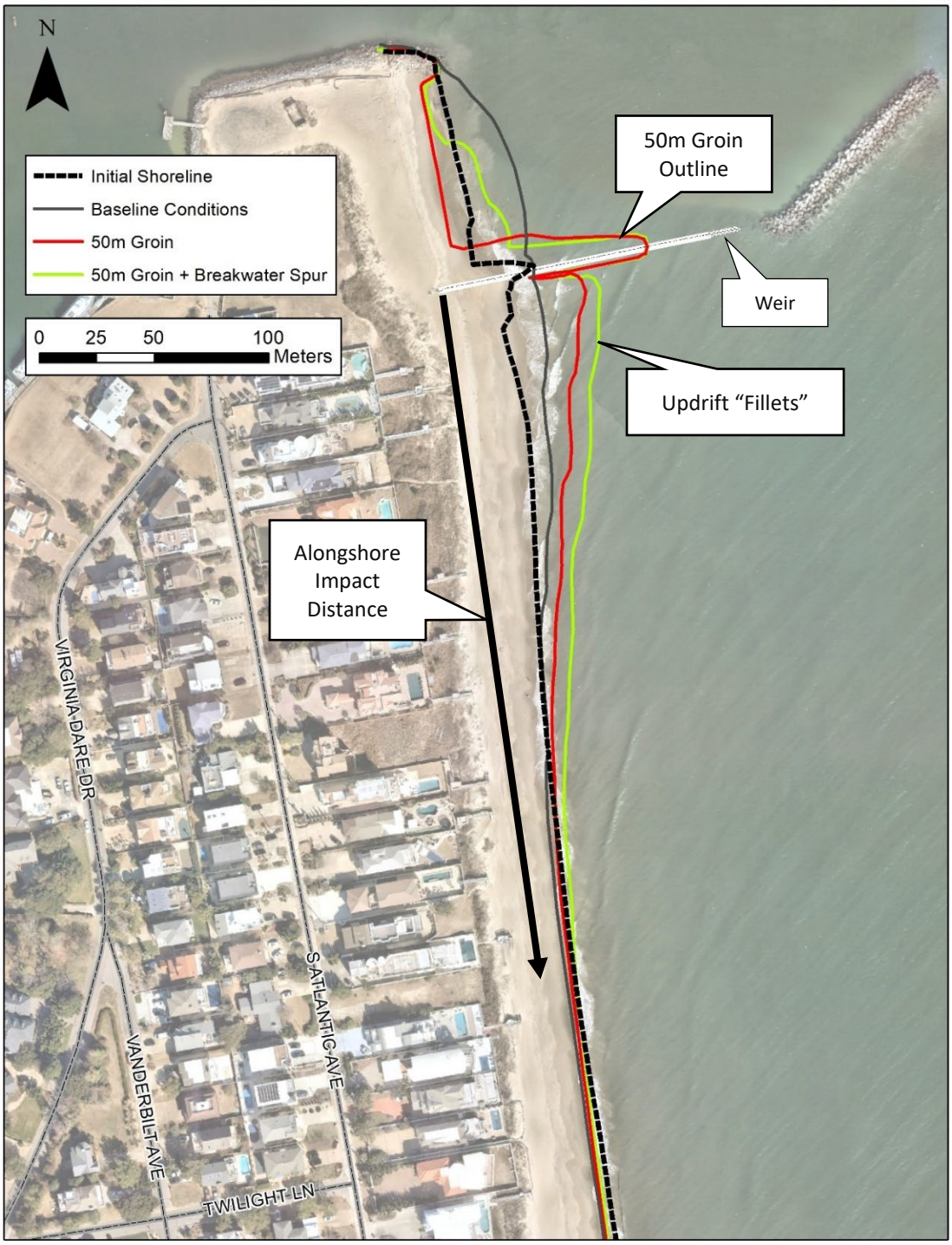


Figure 4-29: Modeled 0m NAVD contour location for Case 7 (50m groin + breakwater spur) over a one-year simulation.

Case 7 had additional positive impacts to beach width and alongshore sedimentation relative to Case 4. Relative to Case 4, Case 7 added additional beach width, ~70m (~230 ft) of additional alongshore benefit distance for the 0m NAVD contour, and an additional +6,460 cy of sediment to the updrift fillet. In total, Case 7 provided a significant increase in beach width relative to Baseline Conditions (Figure 4-29), an

alongshore benefit distance for the 0m NAVD contour line of 310 m (~1,000 ft), and +14,810 cy additional sediment added to the updrift fillet relative to Baseline Conditions. Figure 4-29 shows an increase in beach width and alongshore impact compared to Baseline Conditions and compared to Case 4.

When assessing a typical incoming flood tide condition in Figure 4-30 (different timestep as in previous Section 4.3.2.1), it is shown that adding the rock spur extension to the offshore breakwater at times directs currents within the inlet more northward and westward, moving in more of an arc-like pattern in comparison to Case 4 (50m groin only). This may explain the more erosive/less accretional pattern observed in Figure 4-27. Current speeds over the weir see negligible change, but currents updrift of the groin along the shoreline are reduced due to the disruption of currents by the rock spur extension. This helps explain the observed increase in sedimentation west of the spur and south of the weir. As sediment movement is affected by current speeds, the observed decrease in current speeds allows more sediment to accumulate to the west of the structure. This also has a positive effect on the updrift beach (Croatan Beach) where Case 7 shows an increase in beach sediment and beach width relative to Case 4 (50m groin-only).

Figure 4-31 and Figure 4-32 presents Case 7 under typical outgoing ebb tide and flood tide conditions, respectively, when the current speeds over the weir are relatively high. Results reveal that the rock spur extension slightly increases currents directed through the inlet and north, and currents approaching the breakwater from the south headed northeast are reduced and pushed further south. This may explain the observed increase in sedimentation/less erosion to the north at the Resort Beach – instead of going offshore currents are directed more northward due to the disruption of current direction by the rock spur extension.

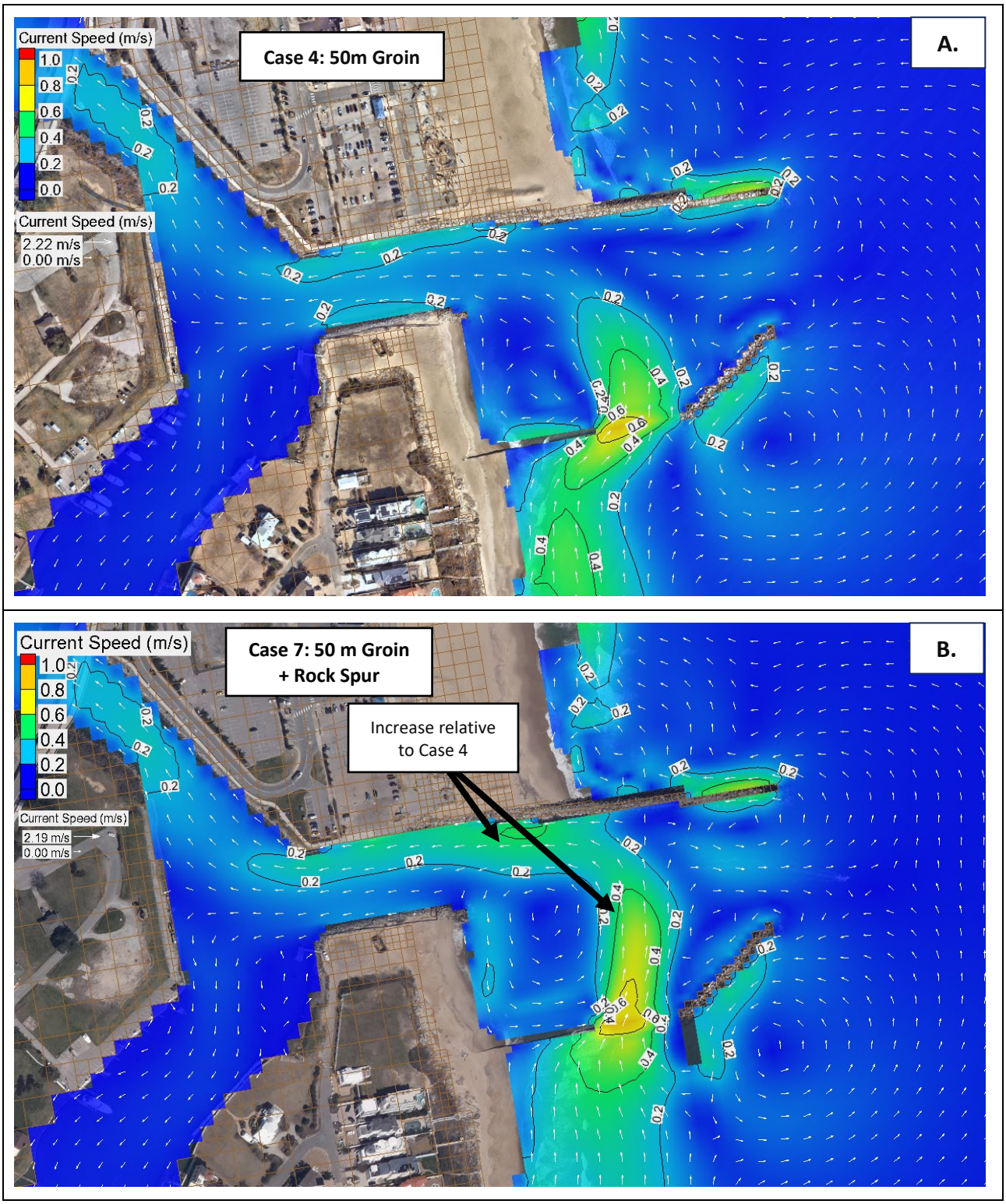


Figure 4-30: Modeled current speeds during a typical flood tide when weir speeds are moderate. A) Case 4: 50m groin; B) Case 7: 50m groin + breakwater spur.

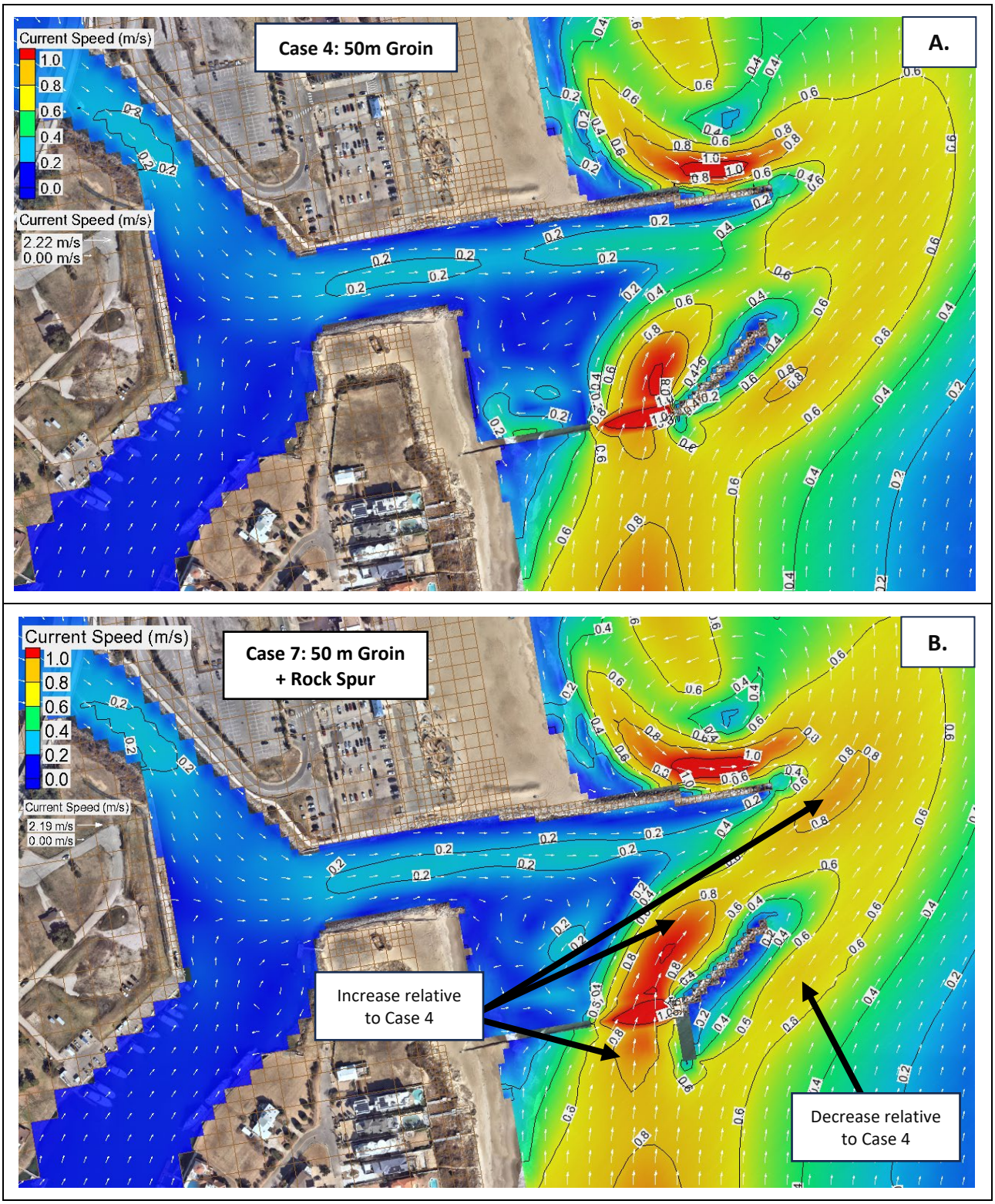


Figure 4-31: Current speeds during a typical ebb tide when flows are high over the weir. A) Case 4: 50m groin; B) Case 7: 50m groin + breakwater spur.

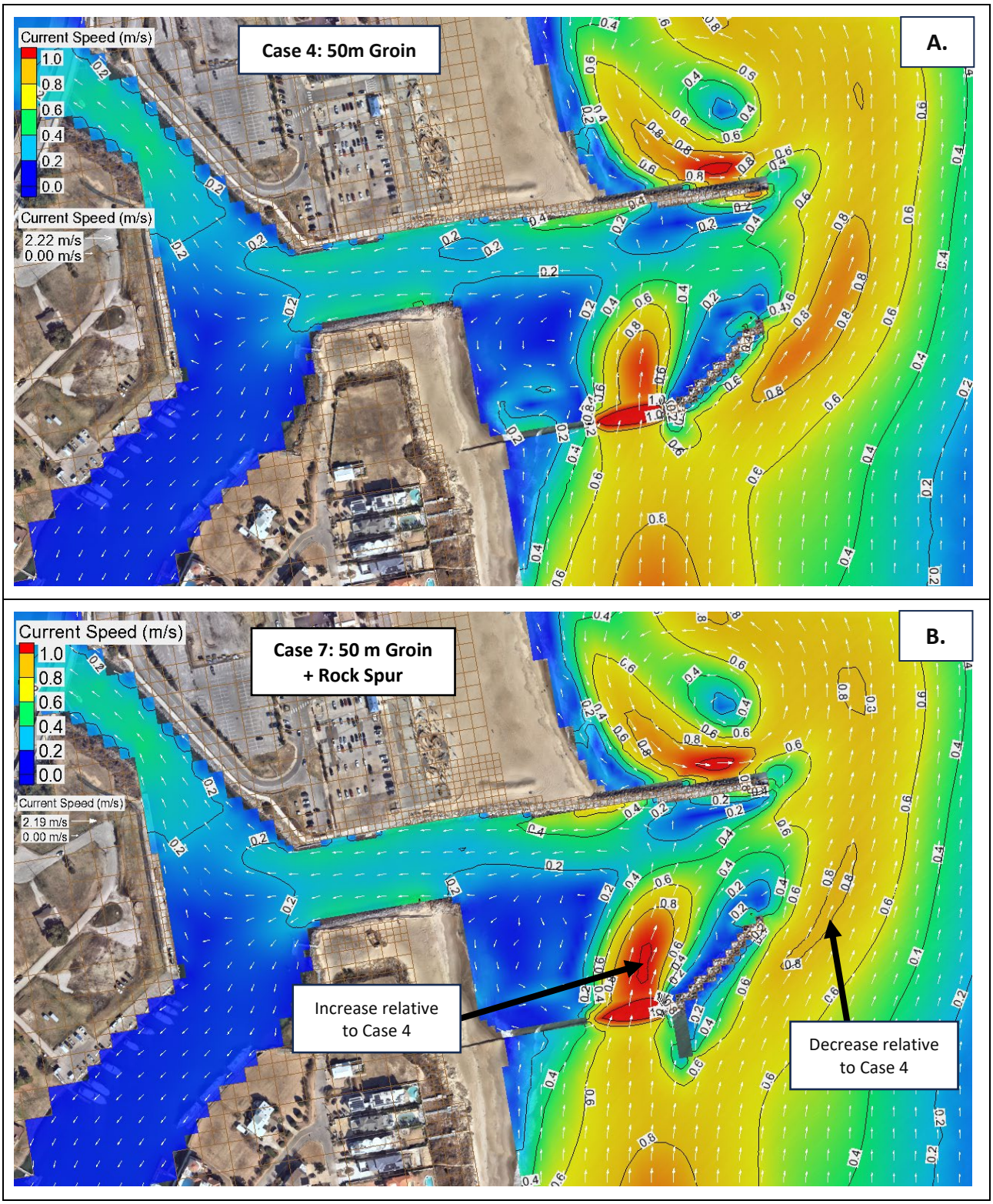


Figure 4-32: Current speeds during a typical flood tide when weir speeds are high. A) Case 4: 50m groin; B) Case 7: 50m groin + breakwater spur.

Examining sediment transport directionality around Rudee Inlet (Figure 4-33), results reveal that when the breakwater spur is included in the model, sediment transport directed northeast over the weir increased.

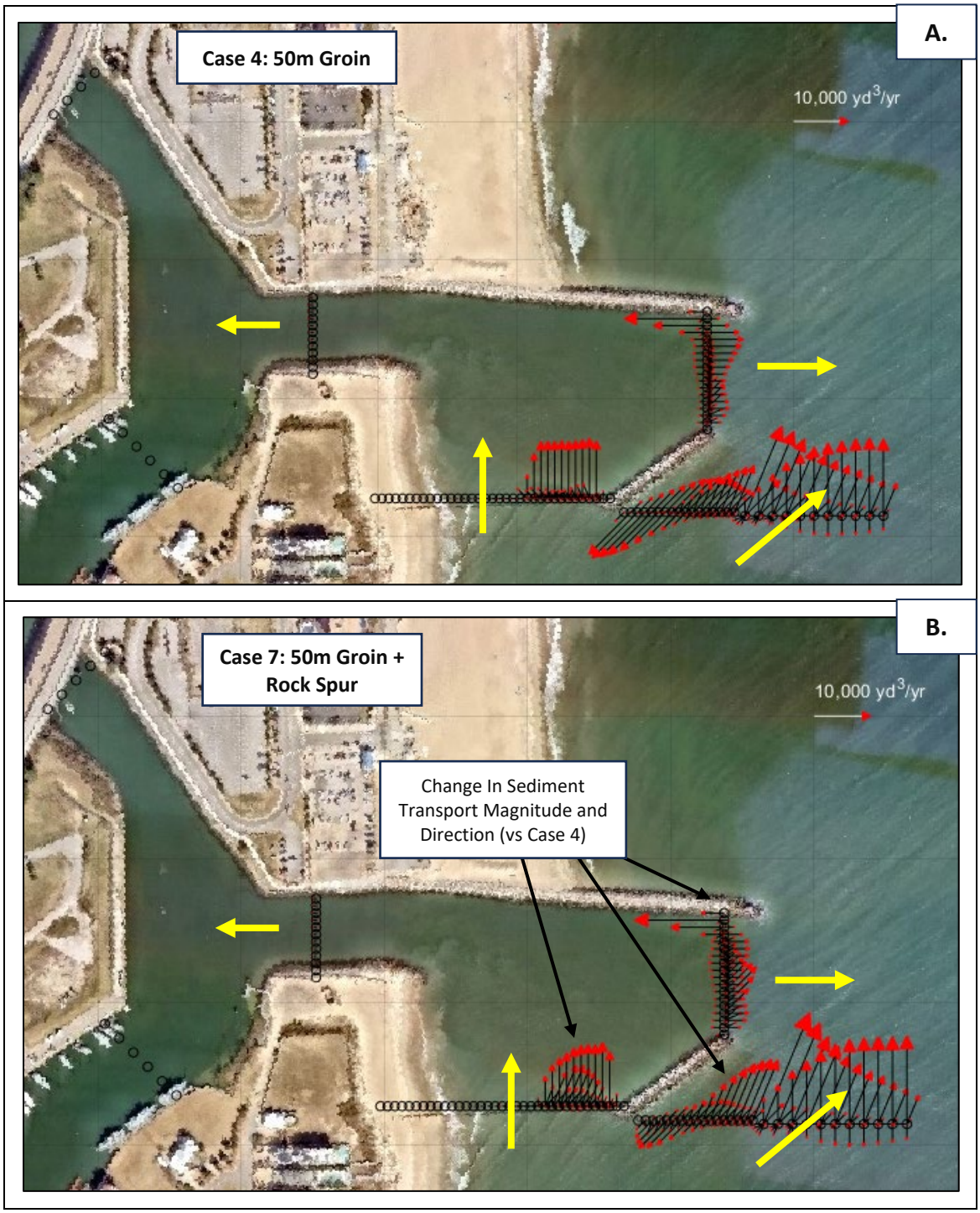


Figure 4-33: Gross sediment transport vectors over the one-year simulation for Case 4 (A.) and Case 7 (B.). Overlaid with the net sediment transport direction for each transect (yellow arrows).

On the seaward side of the breakwater, sediment transport headed southwest decreased, and transport headed northeast was shifted east and redirected more northward. This agrees with the results discussed above regarding current speeds: as flow is redirected due to the breakwater spur, either through the inlet over the weir and out of the mouth, or shifted in its path northeast around the breakwater, sediment transport increases as well.

4.4.2.3 Case 8: South Jetty Extension Results

Case 8 was developed to assess the effectiveness of extending the south jetty along the inlet shoreline within the weir/jetty system. Case 8 also includes the 50-meter groin extension. The primary goal of Case 8 was to drive more sediment accumulation farther east into deeper areas of the sand trap and reduce sediment movement into the inlet throat. In reviewing Case 8 results, the collective amount of sediment that accumulates in the inlet system decreases from 89,130 cy in Case 4 to 84,700 cy in Case 8 (a reduction of 4,430 cy, or 5%). This is still a net benefit relative to Baseline Conditions which accumulated 97,850 cy over a year (total reduction of 13,150 cy, or 13%).

Case 8 was assessed for changes in inlet sedimentation/erosion relative to Case 4 (50m-groin only) to isolate the effect of the additional south jetty rock extension. Figure 4-34 shows the modeled elevation differences for Case 8 relative to Case 4 at the end of the 1-year simulation. As shown in Figure 4-34, the sedimentation patterns vary within and around the inlet system. Results reveal that with the addition of the rock groin extension at the south jetty (in addition to the 50m groin), there is more sedimentation in the sand trap area and onto the shoreline downdrift of the weir, and less sedimentation in the inlet throat and Lake Rudee entrance channel, as expected.

However, there are areas of the inlet system that show a relative increase in sedimentation relative to Case 4 outside of the sand trap, namely in the northern section of the inlet throat and along the inlet side of the north jetty. At the northern side of the inlet throat, the increase in elevation is due to a combination of a decrease in erosion as well as an increase in sedimentation under the proposed groin scenario. This is likely due to sediment moving around the inlet due to the change in currents entering and exiting the inlet. Overall, there is a net decrease in sediment accumulating in the inlet with the south jetty rock extension relative to both Case 4 and Baseline Conditions, and therefore a net benefit to adding the rock extension to the inlet system. There may be more sediment accumulating on the northern side of the inlet throat due to sediment settling earlier and/or less sediment movement through the throat compared to Case 4. There is some spatial variability in sedimentation patterns in areas offshore to the east and at the inlet mouth, but changes are negligible.

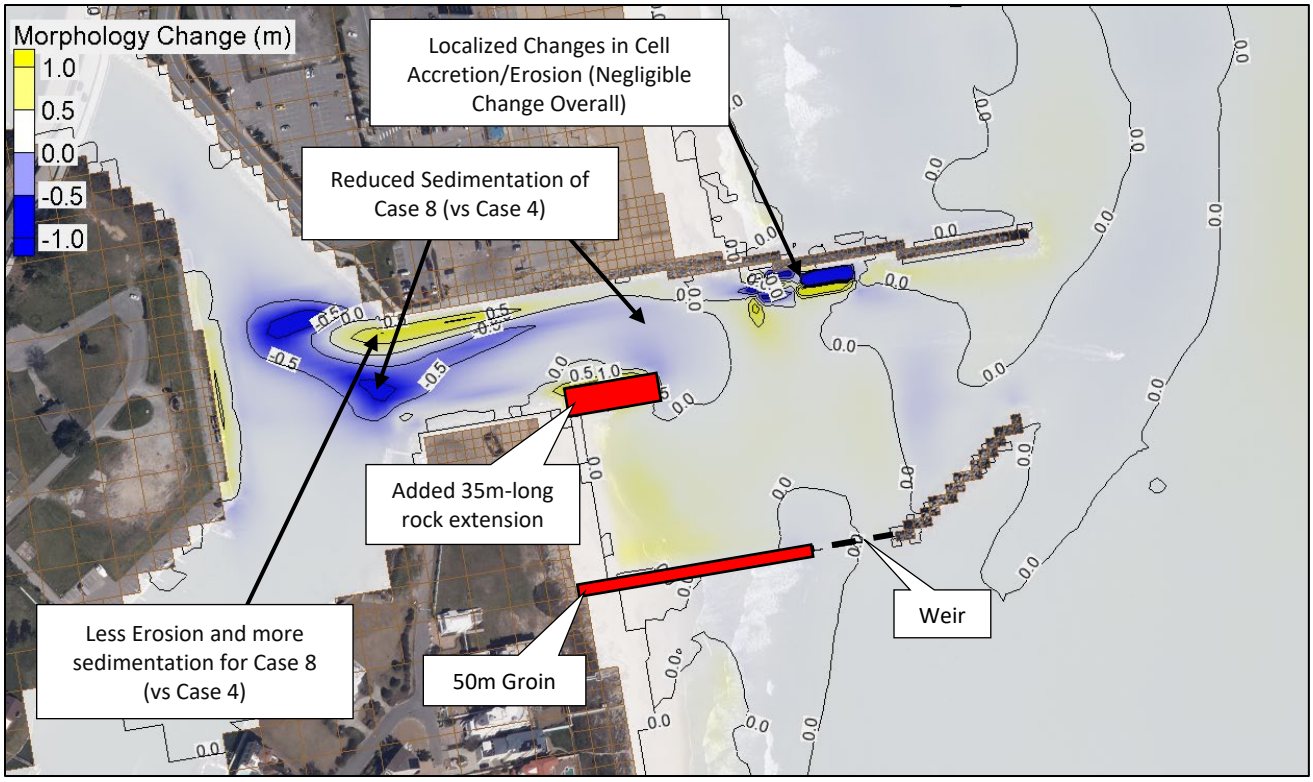


Figure 4-34: Morphology change due to the rock extension off the south jetty relative to Case 4 (50m groin only). Areas of more sedimentation (or less erosion) are shown in yellow, and areas of more erosion (or less sedimentation) are shown in blue. Areas of negligible change are shown in white.

Case 8 had negligible impacts to beach width and alongshore accretion relative to Case 4. Relative to Case 4, extending the south jetty only led to an additional +450 cy of updrift beach accretion, and no additional alongshore benefit distance. One notable difference relative to Case 4 was changes to the downdrift 0m NAVD shoreline relative to the groin, which increased in width due to the additional sedimentation in the area (Figure 4-35).

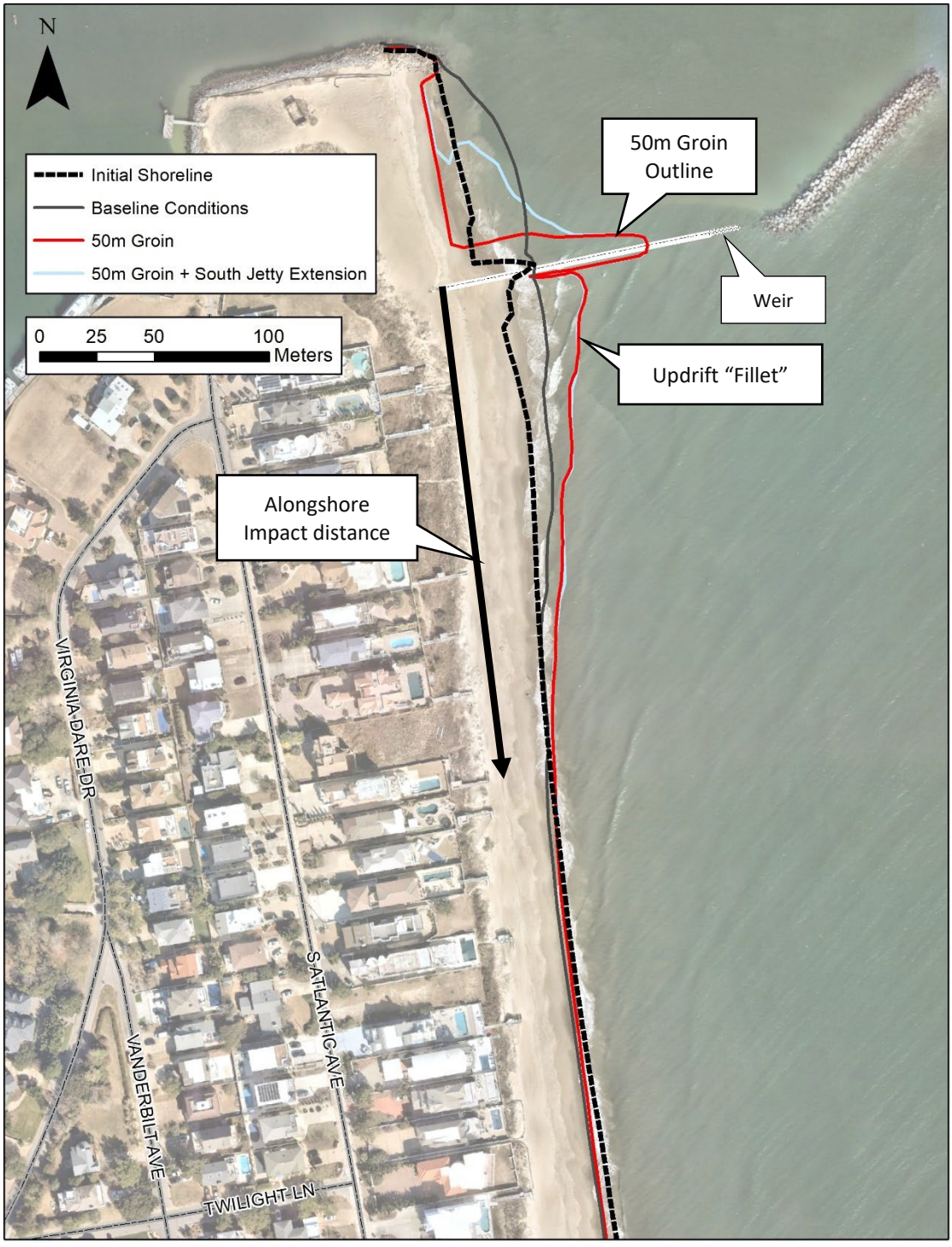


Figure 4-35: Modeled 0m contour location for Case 8 after a one-year simulation.

Figure 4-36 presents Case 8 under a typical flood tide while Figure 4-37 presents Case 8 under a typical ebb tide. Current speeds are minimally affected by the addition of the rock extension off the south jetty relative to Case 4, however, at times current speeds are reduced at the shoreline updrift of the south jetty extension, where additional alongshore accretion is observed relative to Case 4.

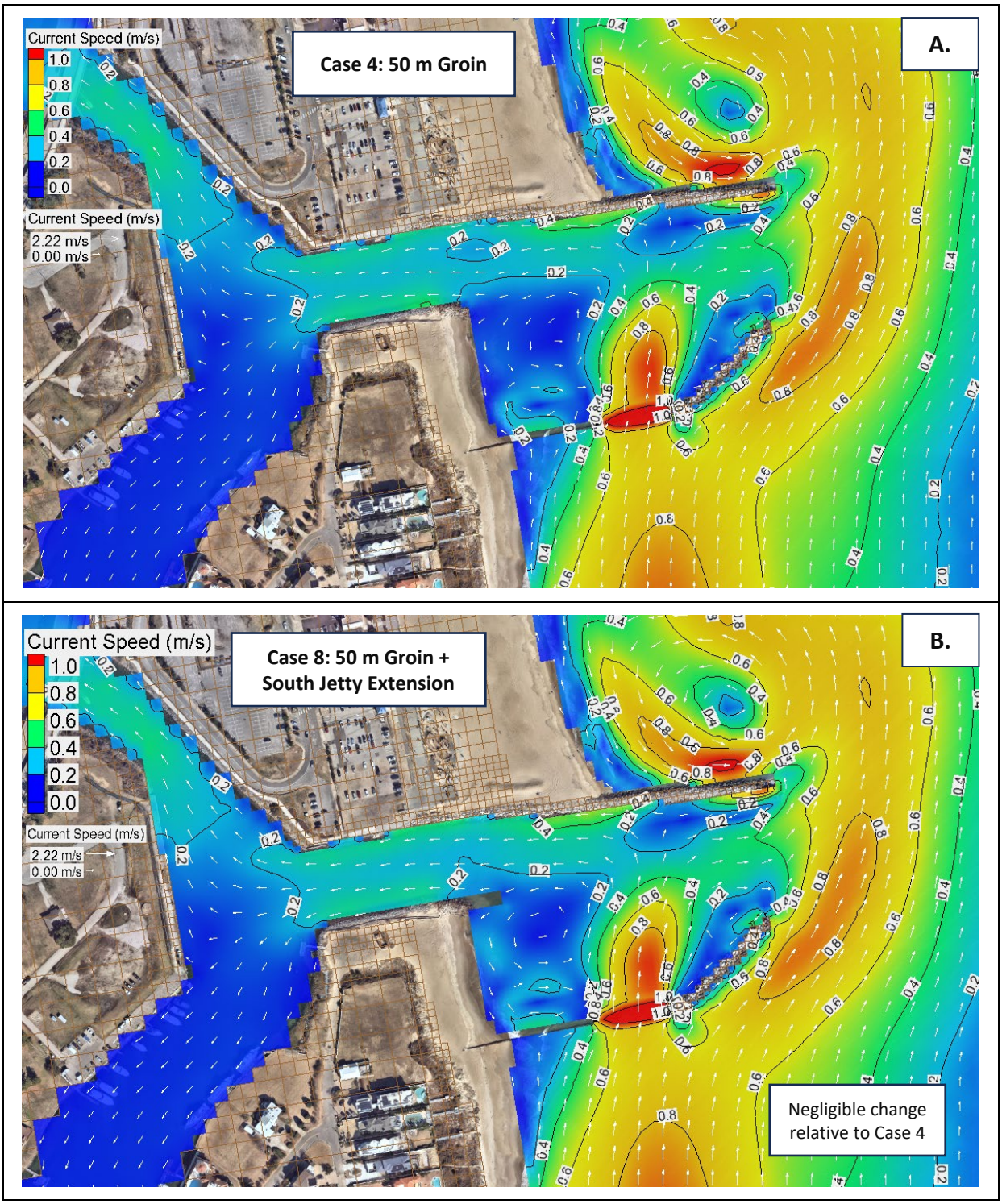


Figure 4-36: Current speeds during a typical flood tide when current speeds are high over the weir. A) Case 4: 50m groin; B) Case 7: 50m groin + south jetty extension.

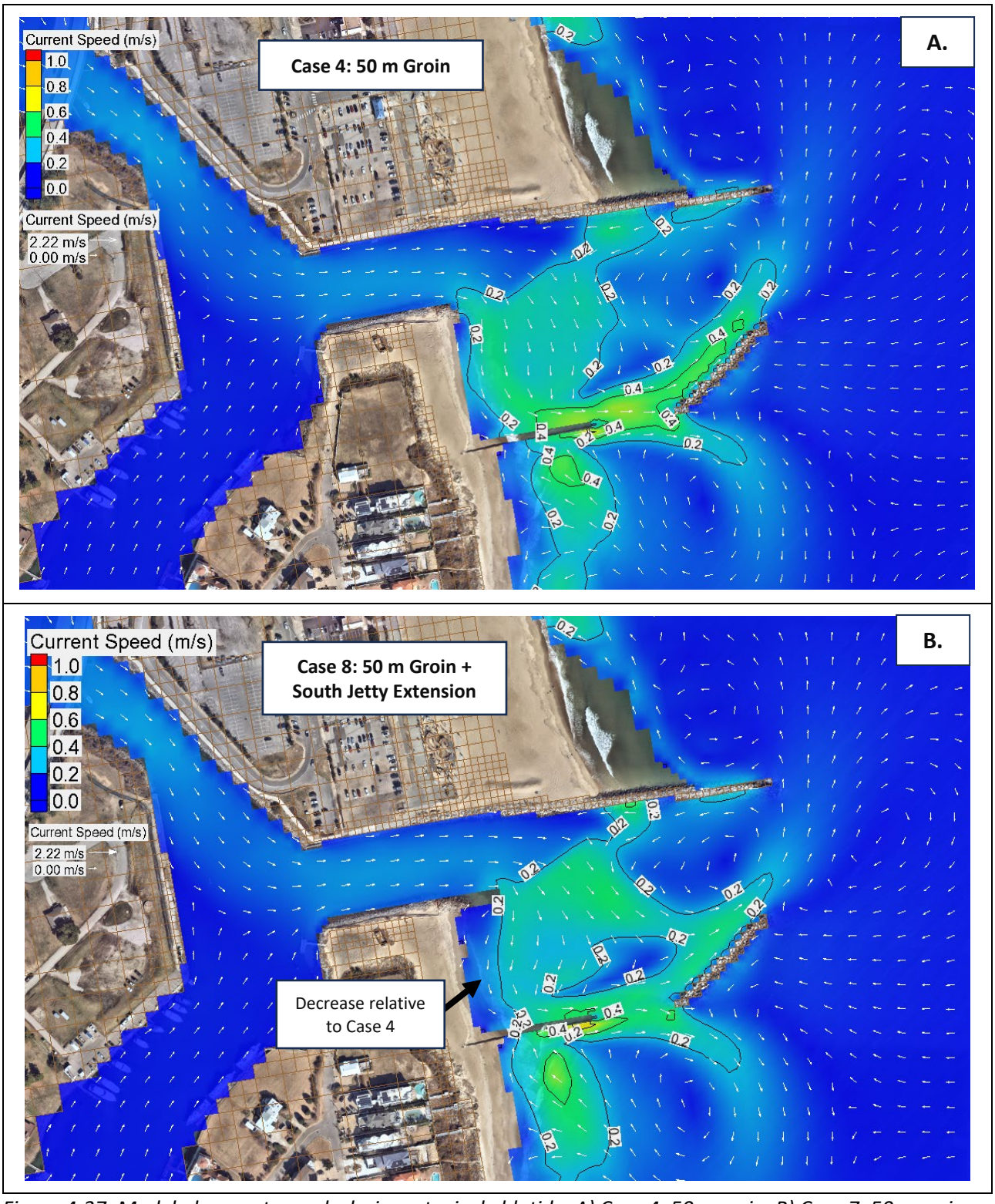


Figure 4-37: Modeled current speeds during a typical ebb tide. A) Case 4: 50m groin; B) Case 7: 50m groin + south jetty extension.

Examining sediment transport directionality around Rudee Inlet (Figure 4-38), results reveal that sediment transport at the inlet is minimally affected for the south jetty extension.

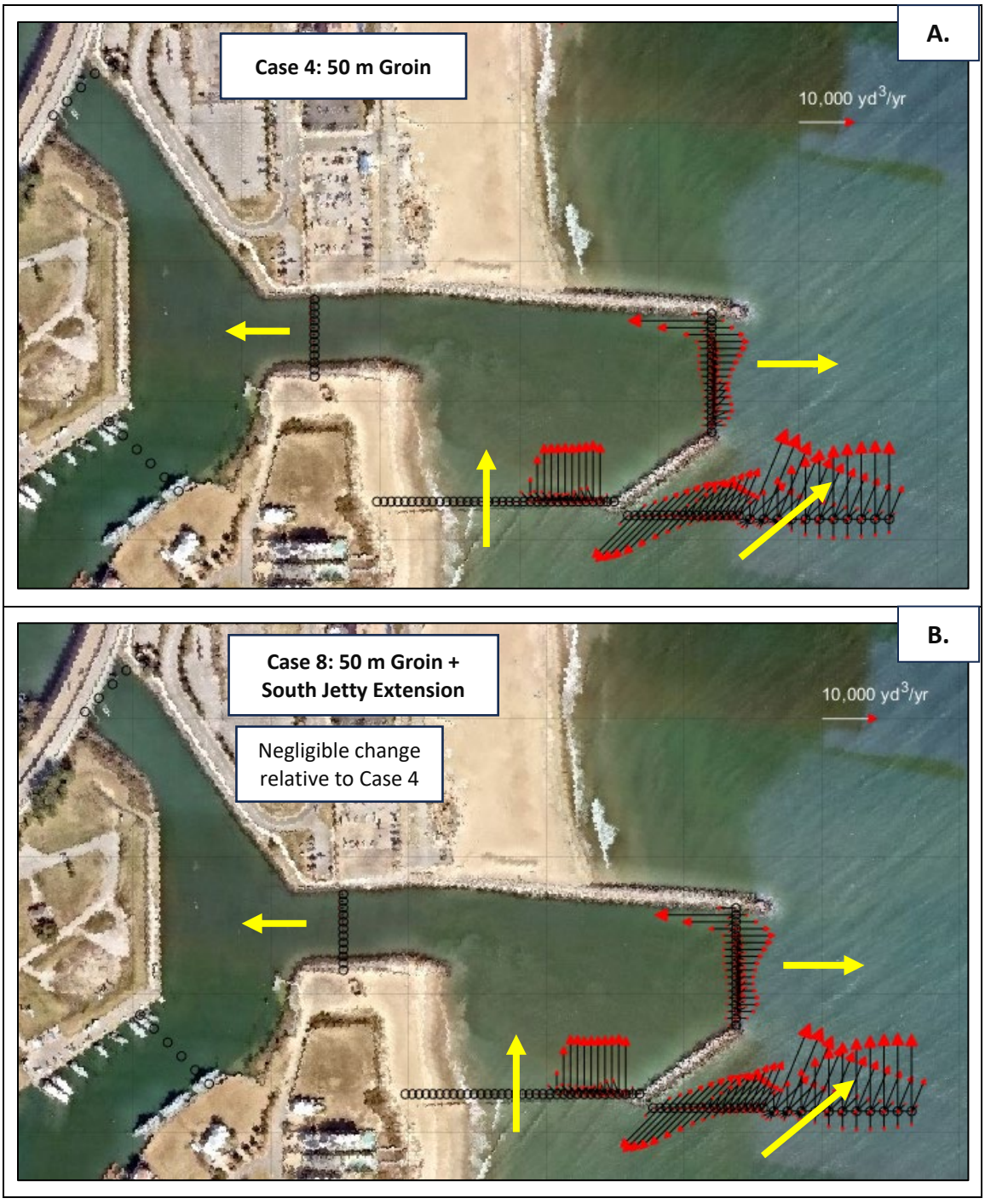


Figure 4-38: Gross sediment transport vectors over the one-year simulation for Case 4 (A.) and Case 8 (B.). Overlaid with the net sediment transport direction for each transect (yellow arrows).

4.4.2.4 Case 9: Raised Weir Results

Case 9 was developed to assess a 0.3 meter increase in the weir elevation. This increase is for the lower, seaward section of the existing weir. The existing weir and the 0.3m increase are both within the MHW/MLW tide range, however the raised weir alternative will reduce the amount of time when water and sediment flow over it. The primary goal of Case 9 was to reduce sediment transport over the weir and increase updrift beach width. In examining the collective amount of sediment that accumulates in the inlet system for Case 9, the total amount of sediment decreases from 89,130 cy in Case 4 to 86,310 cy in Case 9 (a reduction of 2,820 cy, or 3%). This also represents a net benefit relative to Baseline Conditions which accumulated 97,850 cy over a year (total reduction of 11,540 cy, or 12%).

Case 9 was assessed for changes in inlet sedimentation/erosion relative to Case 4 (50m-groin only) to isolate the effect of the additional 0.3m of weir elevation. Figure 4-39 presents the 1-year modeled elevations for Case 9 relative to Case 4. As shown in Figure 4-39, the sedimentation patterns vary within and around the inlet system. Results reveal that when the weir is raised by 0.3 meters in elevation (relative to Case 4), sedimentation is reduced in parts of the sand trap area, inlet throat, and Lake Rudee channel. There is also less sedimentation offshore to the east, indicating that sediment is being redirected elsewhere, either to the north or possibly offshore out of the system. Redirection north is likely due to a greater number of instances when the weir is emergent, thereby redirecting flow to the northeast around the breakwater instead. However, there are areas of the inlet system that show a relative increase in sedimentation relative to Case 4 outside of the sand trap, namely in the northern section of the inlet throat and just north of the weir. This is likely due to sediment moving around the inlet due to the change in currents entering and exiting the inlet. There is some spatial variability in sedimentation patterns at the inlet mouth, but changes are negligible. Overall, there is a net decrease in sediment accumulating in the inlet with the raised weir relative to both Case 4 and Baseline Conditions.

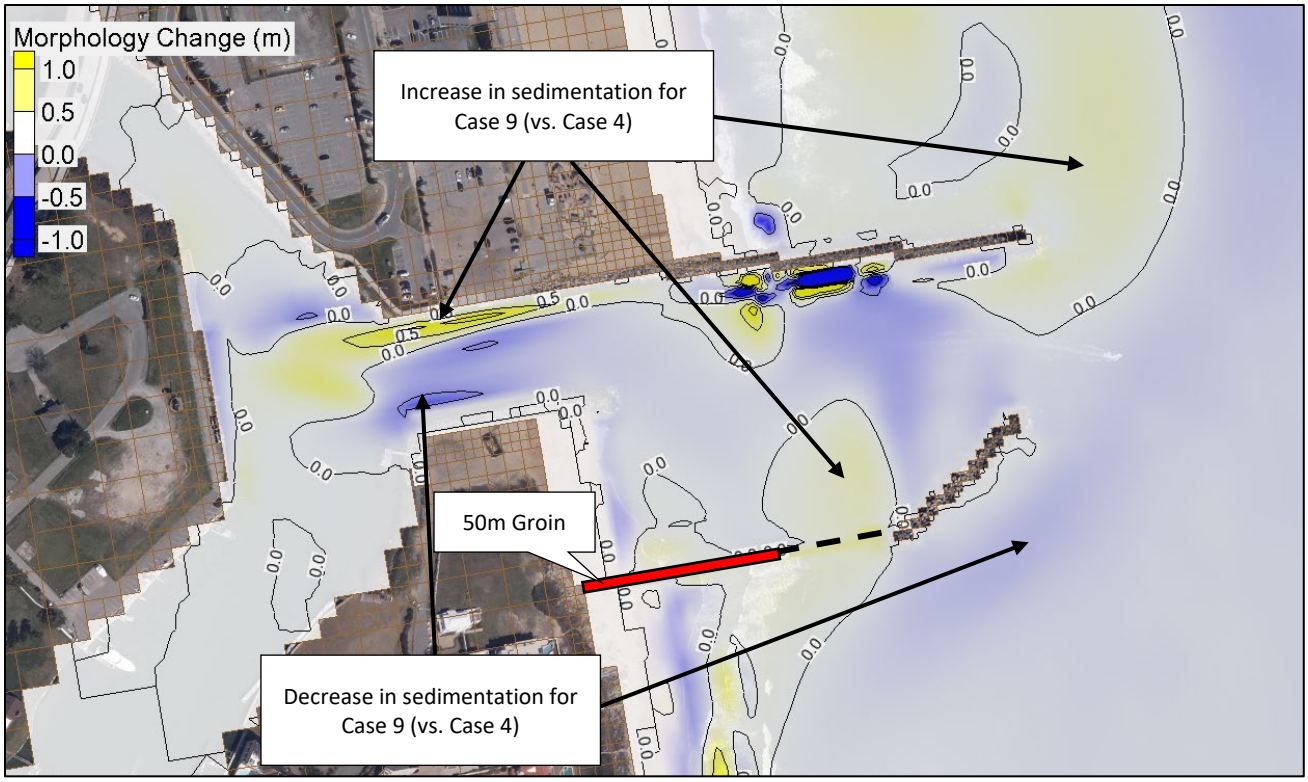


Figure 4-39: Morphology change due to raised weir (Case 9) relative to Case 4 (50m groin only). Areas of more sedimentation (or less erosion) are shown in yellow, and areas of more erosion (or less sedimentation) are shown in blue. Areas of negligible change are shown in white.

Case 9 had additional positive impacts to beach width and alongshore accretion relative to Case 4. Relative to Case 4, Case 9 added additional beach width, ~20m (~65 feet) of additional alongshore impact distance for the 0m NAVD contour, and an additional +1,210 cy of sediment to the updrift fillet. Relative to Baseline Conditions, Case 9 provided significant increases in beach width, a total alongshore impact distance for the 0m NAVD contour line of 260 m (~850 feet), and +9,560 cy additional sediment added to the updrift fillet. Figure 4-40 shows the modeled 0m contour for Case 9 and the associated increase in beach width and alongshore benefit compared to Baseline Conditions and Case 4.

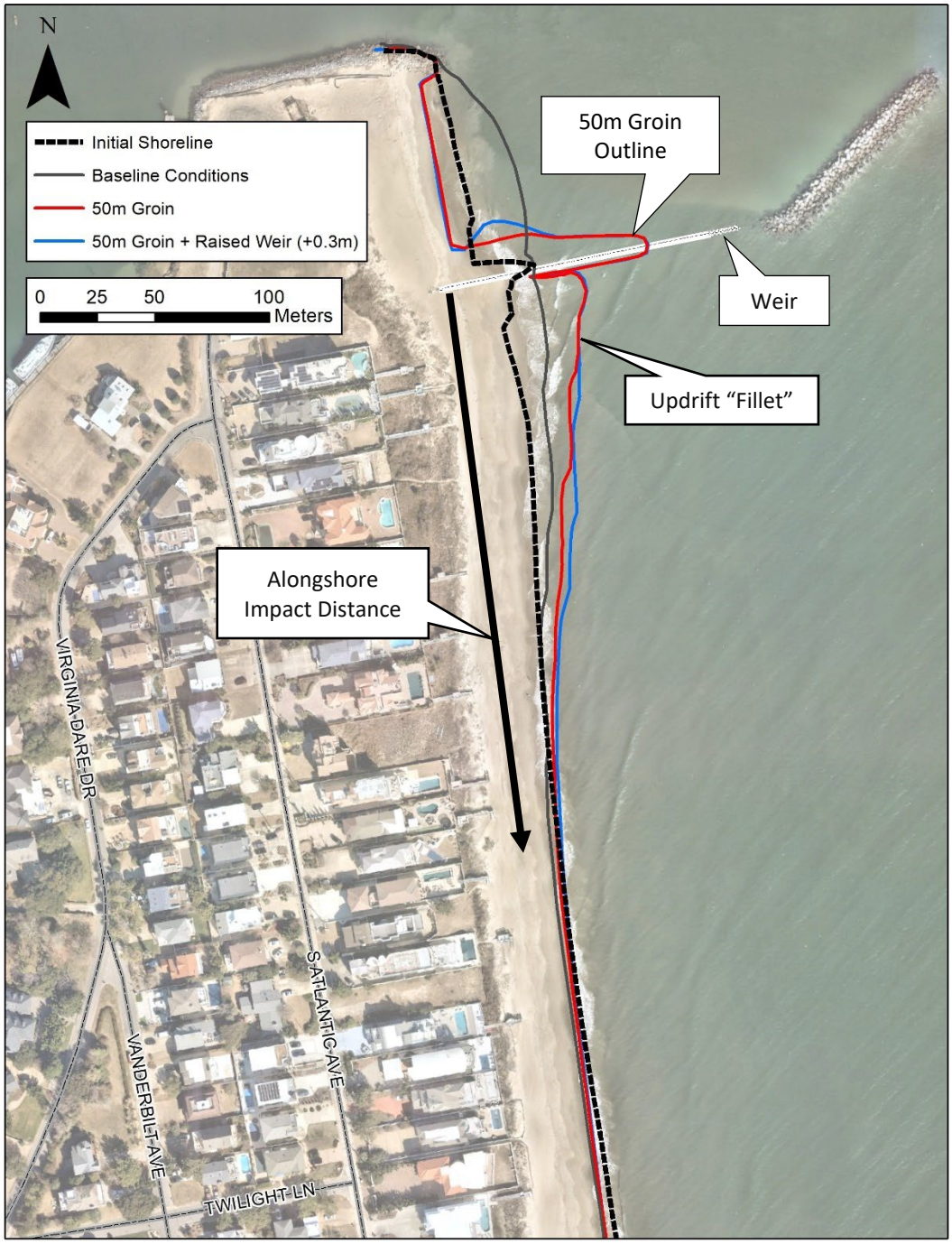


Figure 4-40: Modeled 0m NAVD contour location or Case 9 after one-year simulation.

Figure 4-41 presents typical flood and ebb snapshots for Case 9 and compares them with Case 4. Results reveal flow over the weir is reduced due to the raised elevation and instead is redirected northeast around the breakwater. When the weir is emergent ("dry"), flow is restricted from entering the inlet through the weir – this either redirects currents to the northeast around the breakwater (Figure 4-42) or redirects current speeds to the southeast (Figure 4-43), depending on the tide and current direction.

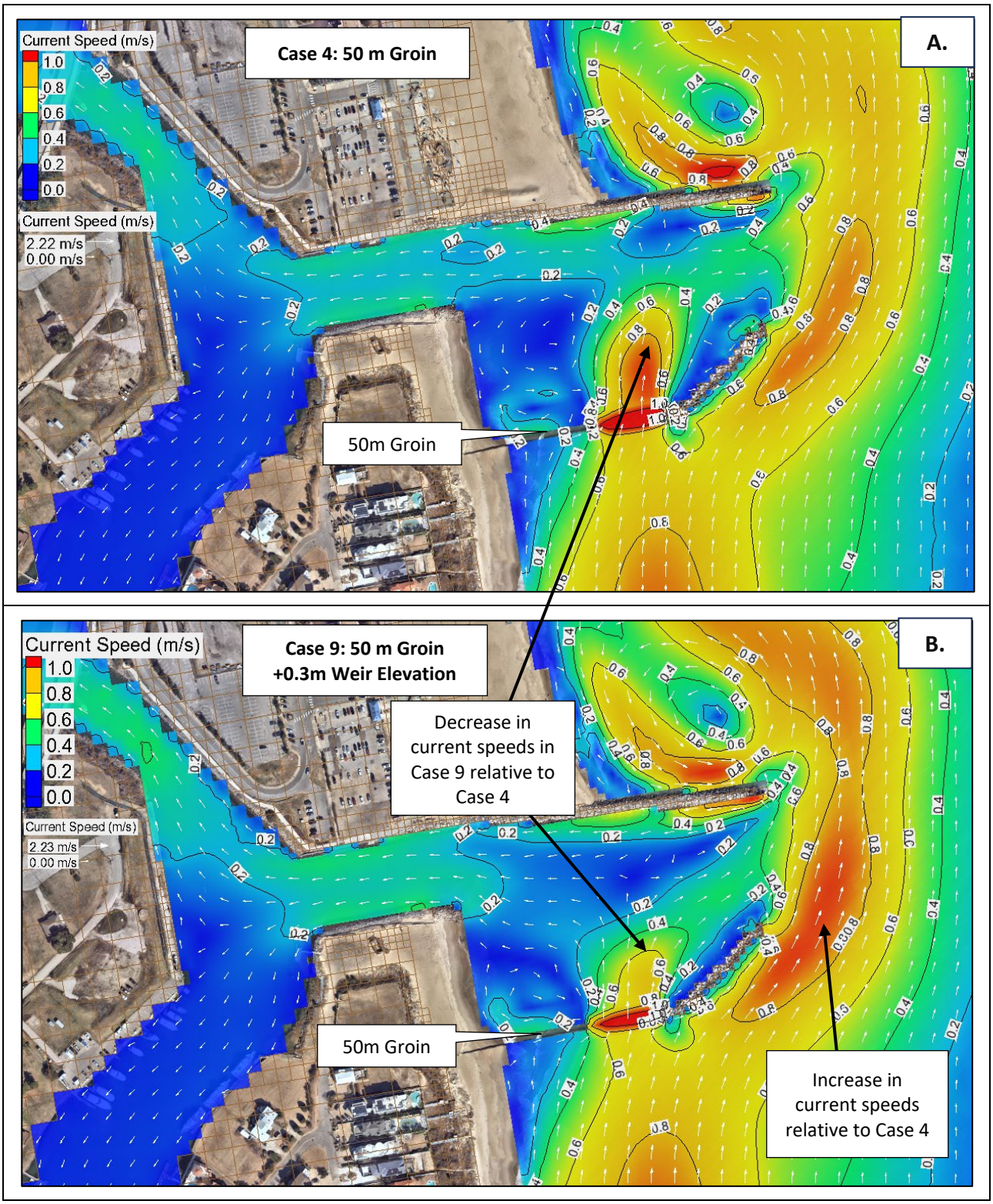


Figure 4-41: Current speeds during a typical flood tide when current speeds over the weir are high. A) Case 4: 50m groin; B) Case 9: 50m groin + raised weir.

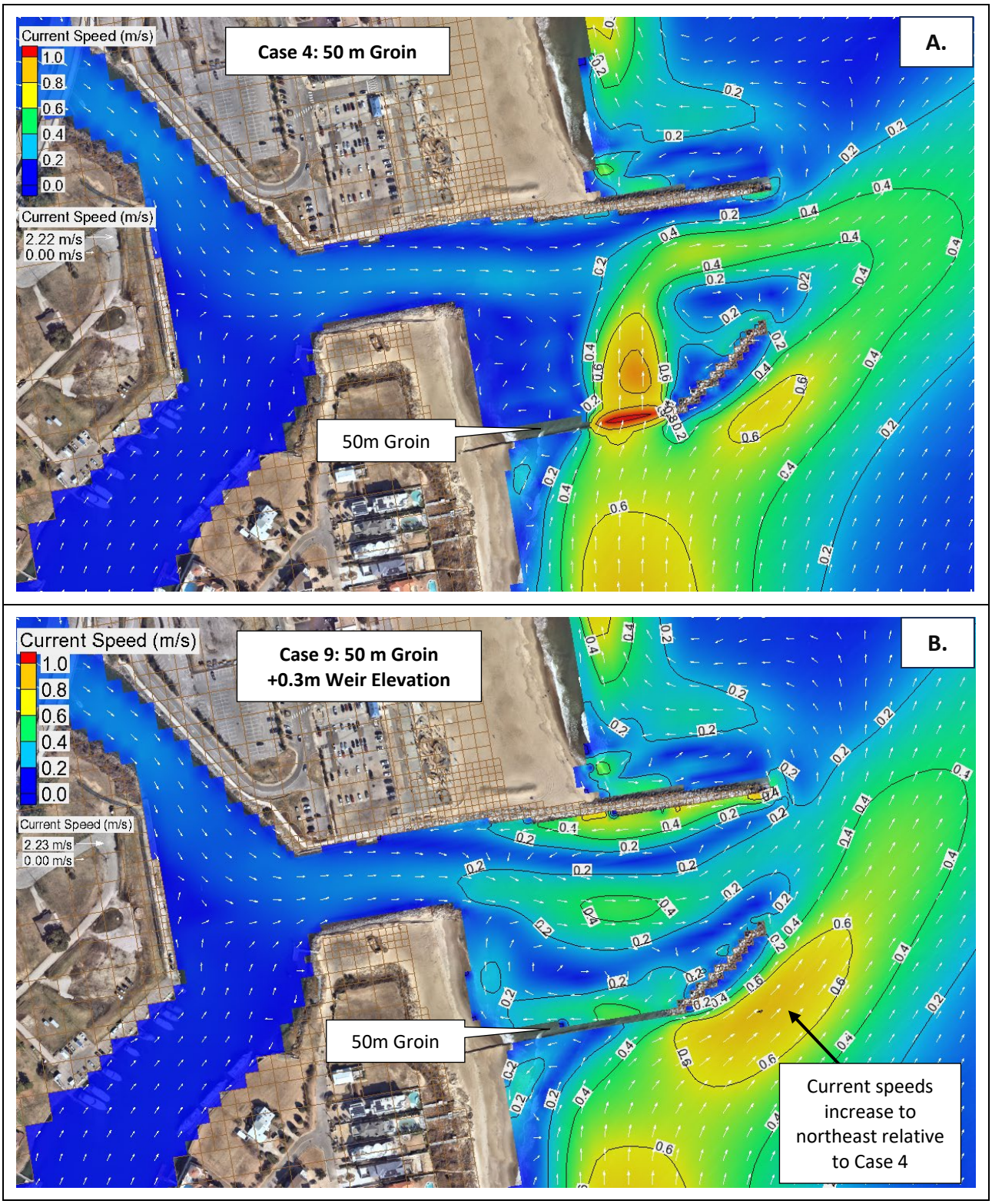


Figure 4-42: Case 9 current speeds during a typical ebb tide when the weir is emergent for Case 9 (relative to Case 4). A) Case 4: 50m groin; B) Case 7: 50m groin + raised weir.

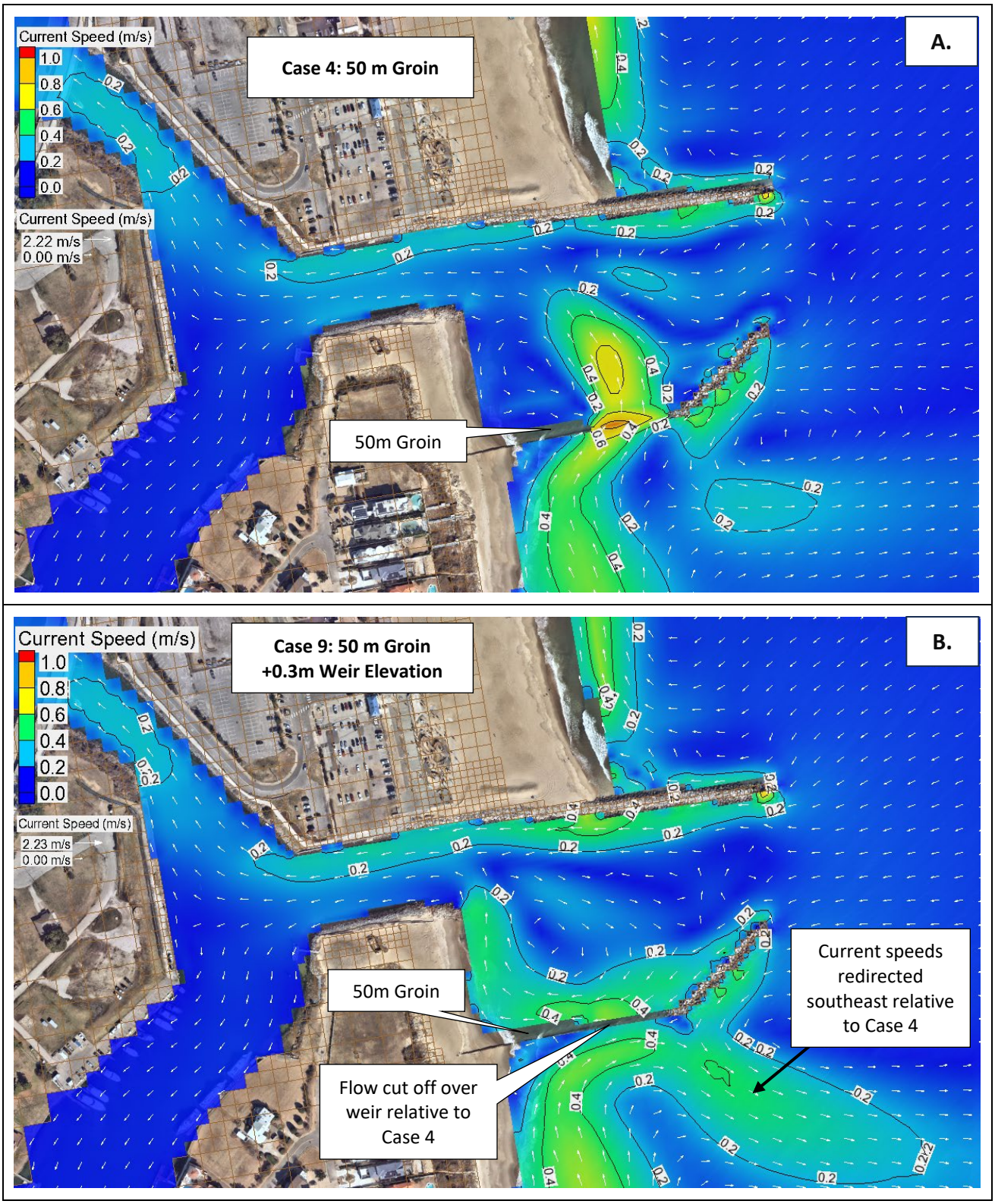


Figure 4-43: Current speeds during a typical flood tide when the weir is emergent (relative to Case 4). A) Case 4: 50m groin; B) Case 9: 50m groin + raised weir.

Figure 4-44 presents sediment transport vectors for Case 9 in comparison to Case 4. Sediment transport magnitudes and directionality around Rudee Inlet reveal that the additional weir elevation decreases sediment transport over the weir while there is a slight decrease in sediment leaving the inlet through the inlet mouth, and a slight increase in sediment transport directed northeast along the breakwater. This is primarily due to a decrease in flow and current speeds over the weir and out of the inlet, as well as an increase in flow directed northeast along the breakwater as discussed previously.

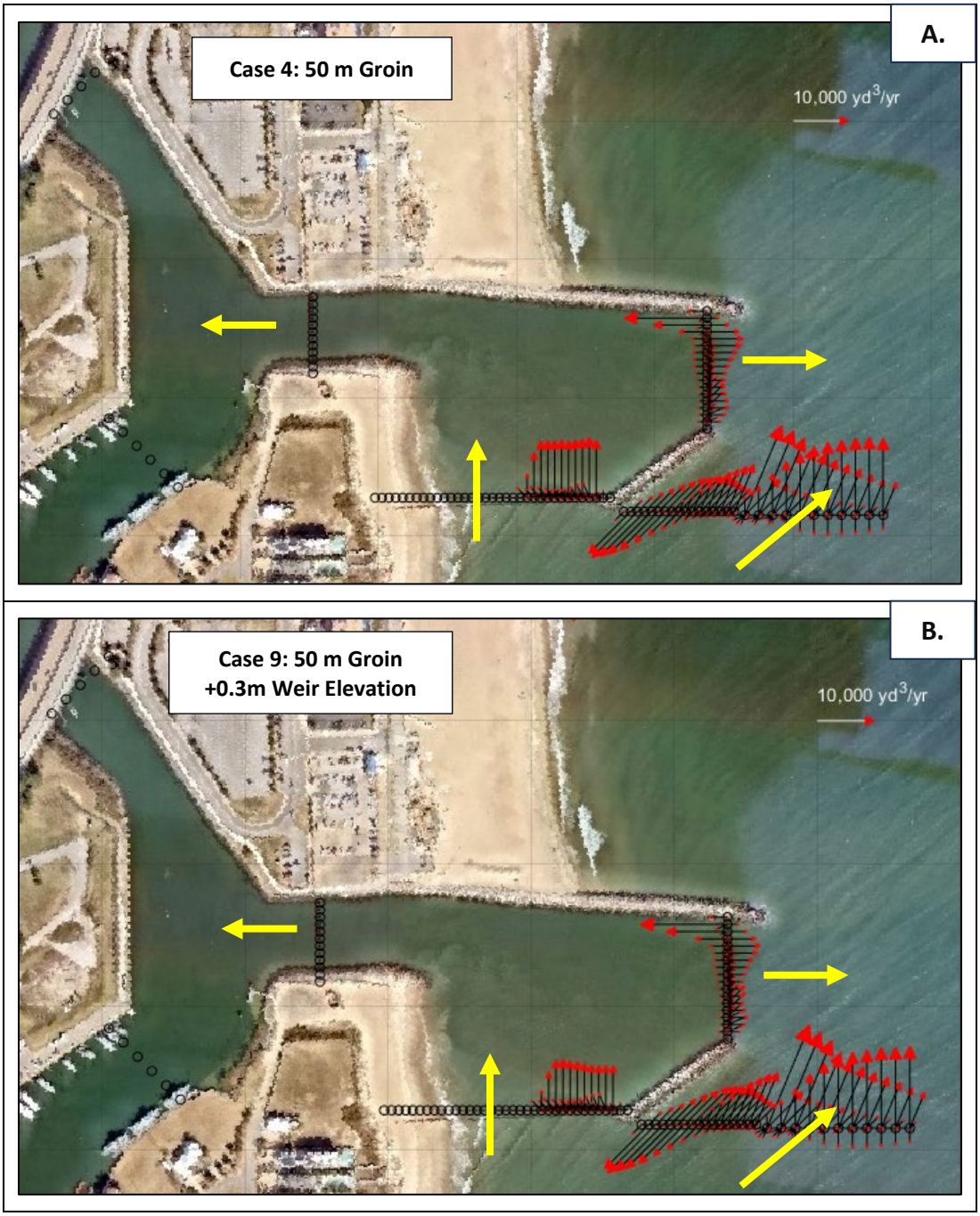


Figure 4-44: Gross sediment transport vectors over the one-year simulation for Case 4 (A.) and Case 9 (B.). General net sediment transport direction for each transect is indicated with yellow arrows.

5.0 Summary and Recommendations

Geosyntec (doing business as ATM), was contracted by the City of Virginia Beach, VA (Client) to evaluate the potential improvements to the existing Rudee Inlet structures and to evaluate their effects on hydrodynamics and sediment dynamics in the system. In particular, the study focused on optimizing the weir/rock groin lengths to increase the updrift recreational beach width without adversely impacting the rate of sediment inflow into the inlet and in turn, dredging frequency. A secondary goal of the model was to assess the impact of additional rock structures elsewhere in the inlet system to improve dredging efficiency.

The Coastal Modeling System (CMS) coupled model suite (CMS-Wave and the CMS-Flow hydrodynamic and sediment transport model) was set up and run to simulate hydrodynamic, wave, and sediment transport process in the project study area. A 1-year simulation representative of typical site conditions was run to model and assess inlet improvement scenarios under identical conditions.

Data collection to support model calibration was conducted and included the following:

- Bathymetry
- Water levels
- Currents
- Flows

Several other publicly available sources of data were also incorporated into the modeling effort.

Overall, the existing weir is proposed to be shortened by extending the existing rock groin on the weir's landward side. Results reveal that a rock groin extension (which in turn shortens the weir and shifts the weir transport pathway farther seaward) will improve the functioning of the jetty/weir system. The seaward shift in the weir sediment pathway also facilitates dredging of the sand trap system.

The three primary goals of assessing the groin and additional rock extension scenarios were to:

1) Increase updrift recreational beach width at Croatan Beach
2) Maintain or reduce sedimentation in the inlet system by reducing inflow over the weir
3) Improve dredging efficiency

When evaluating all Cases together, the relative benefit derived from each rock extension scenario can be identified.

Examining the first goal of increasing updrift beach (Croatan Beach) width and length, all modeled groin extension Cases (Cases 1-6) resulted in an increase in beach width relative to Baseline Conditions. There was additional benefit to updrift beach accretion with additional rock extensions to the breakwater (Case 7) and a raised weir (Case 9).

Examining the second goal of maintaining or reducing the total accumulated sediment in the inlet system, all modeled Cases showed a net decrease in sedimentation in the inlet system under all groin scenarios. Therefore, less dredging effort is anticipated relative to baseline conditions under longer groin extension lengths.

Conversely, results revealed that as the groin was lengthened, the amount of sediment that collected along the outer reaches of the inlet and further north at the Resort Beach increased. Additional sedimentation at the inlet mouth due to the groin may present additional challenges, as sedimentation in this area may exacerbate shoaling issues for vessel traffic. Therefore, selection of the preferred groin alternative for design should be considered a balance between the reduction of sediment accumulation in the inlet, and an increase in sedimentation east of the inlet mouth due to changes in current patterns. Case 7-9 did not exhibit a significant change in the amount of sedimentation at this location compared to Case 4 (50m-groin only scenario).

In terms of weir flow, no significant increases in weir flow were observed due to the extension of the rock groin (and corresponding shortening of the weir) or other rock extensions (e.g., breakwater spur, south jetty extension) in the inlet system.

Examining the third goal of improving dredging efficiency, results revealed that with the addition of the breakwater spur (Case 7), sediment was minimized going offshore and instead redirected towards the weir and Croatan Beach, resulting in more sedimentation updrift of the proposed extended rock groin and more sediment naturally bypassing to the Resort Beach. With the addition of the south jetty extension, sediment accumulation was redistributed more to the sand trap area and reduced sediment accumulation in the inlet throat.

In summary, the modeling effort supports the extension of the rock groin as the primary design priority. The modeling also shows additional benefits to updrift accretion and inlet sedimentation with additional rock extensions added to the inlet system, including a rock spur extension to the landward edge of the existing breakwater and a rock extension to the existing south jetty.

6.0 References

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APPENDIX B
Model Coefficients

Model Input Variable Name	Units	Default Value	Model Input Value	Reasoning
Flow Grid Inputs				
Ramp duration	hrs	24	24	
Hydrodynamic time step	sec	User specified	600	Balanced model stability and accuracy
Water temp	C	15	15	
Water density	kg/m^3	1025	1025	
Solution Scheme	NA	Implicit	Implicit	
Momentum Equation advective terms	NA	ON	ON	
Momentum Equation mixing terms	NA	ON	ON	
Wall friction	NA	ON	ON	
Bed friction	mm	Mannings N: 0.025	Mannings N: 0.02	Increased sediment transport/sedimentation in inlet
Sediment Inputs				
Transport capacity formula	NA	Lund-CIRP	Lund-CIRP	
Sediment density	kg/m^3	2650	2650	
Bed load scaling factor	NA	1	1	
Suspended load scaling factor	NA	1	1	
Morphologic acceleration factor	NA	1	1	
Bed slope coefficient	NA	1	1	
Sediment porosity	NA	0.4	0.4	
Transport grain size	mm	0.2	0.2	
Total load adaptation length method	NA	Maximum of Bed & Suspended Sediment	Constant	Based on previous modeling experience
Total load adaptation length	m	10	10	
D50	mm	0.2	0.24	Matched expected grain size at site

Wave Grid inputs				
Forward reflection	NA	OFF	ON; Spatially constant: 0.5	Accounted for reflection on inlet structures
Backward reflection	NA	OFF	OFF	
Muddy bed	NA	OFF	OFF	
Wave breaking formula	NA	OFF	ON; formula: Extended Goda	Accounted for wave breaking in shallow water
Bed friction	NA	NONE	NONE	
Matrix solver	NA	Gauss-Seidel; Processes: 1	Gauss-Seidel; Processes: 1	
Allow wetting and drying	NA	ON	ON	
Non-linear wave effect	NA	OFF	OFF	
Infragravity wave effect	NA	OFF	OFF	
Diffraction intensity	NA	ON: 4	ON: 4	
Runup	NA	OFF	OFF	
Currents	NA	OFF	OFF	
Spatial wind field	NA	OFF	OFF	
Spatial surge	NA	OFF	OFF	
Wave source	NA	Spectra and wind	Spectra	Wind input not required