APPENDIX B:

ENGINEERING APPENDIX

COLLIER COUNTY COASTAL STORM RISK MANAGEMENT (CSRM) STUDY

JULY 2020

US Army Corps of Engineers®

Norfolk District

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CHAPTER 1 INTRODUCTION

1.1. DESCRIPTION OF PROJECT AREA AND VICINITY

The Collier County Coastal Storm Risk Management (CSRM) Study is a study conducted by the U.S. Army Corps of Engineers, Norfolk District and the Local Sponsor, Collier County, to reduce risk of coastal storm damages and impacts to the year 2080 for two project areas near the cities of Naples and Marco Island. Collier County is located on the southwest coast of Florida, about 120 miles south of the entrance to Tampa Bay and about 100 miles north of Key West. (Figure 1.1**).**

Figure 1.1. Image of Collier County's location (in red) along the coast of Florida.

Figure 1.2 and Figure 1.3 show the currently proposed project areas near the cities of Naples and Marco Island. The most notable beach areas area labeled in Figure 1.3, however the North County consists of all of the beaches north of Gordon Pass along with adjacent inland bay areas. It extends roughly 17 miles from the northern county line (approximately Bonita Beach Road extended) south to Gordon Pass. Collier County currently maintains nourishment projects at Vanderbilt Beach from Florida Department of Environmental Protection (DEP) range monument R22-R30; at Park Shore Beach from R42-54; and Naples Beach from R58A-R79. Other key features of the North County project area are the Barefoot Beach County Preserve (R9-R16), Wiggins Pass State Park (R17-R22), and Naples Pier (R74.5). Within the North County there are four inlets: Wiggins Pass, Clam Pass, Doctors Pass, and Gordon Pass. All four of these inlets are periodically dredged and maintained by the County. Both Wiggins Pass and Clam Pass are natural inlets while Doctors Pass and Gordon Pass were stabilized with jetties in the 1960s.

Marco Island is the largest barrier island within Southwest Florida's Ten Thousand Island area. It

beach spans from Big Marco Pass in the north to Caxambas Pass. The CSRM beach project area extends from the southern tip of a natural spit that is approximately adjacent to the northern terminus of Seaview Ct. (R135), south down the shoreline to the point that bounds the western end of Caxambas Pass. The approximate shoreline length of this project area is three miles.

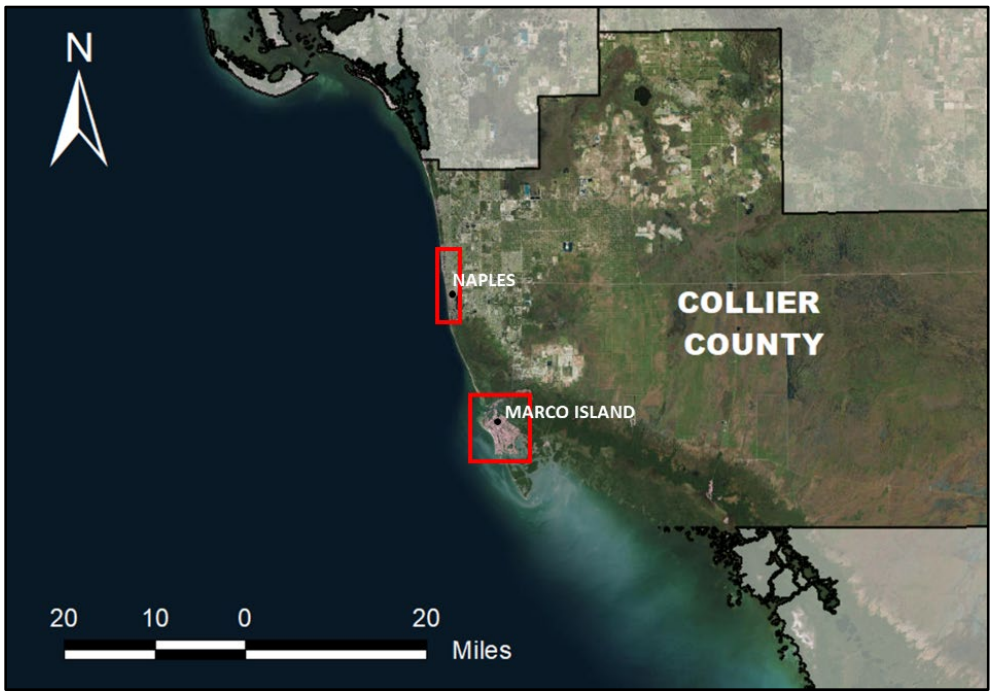


Figure 1.2. Project areas near cities of Naples and Marco Island

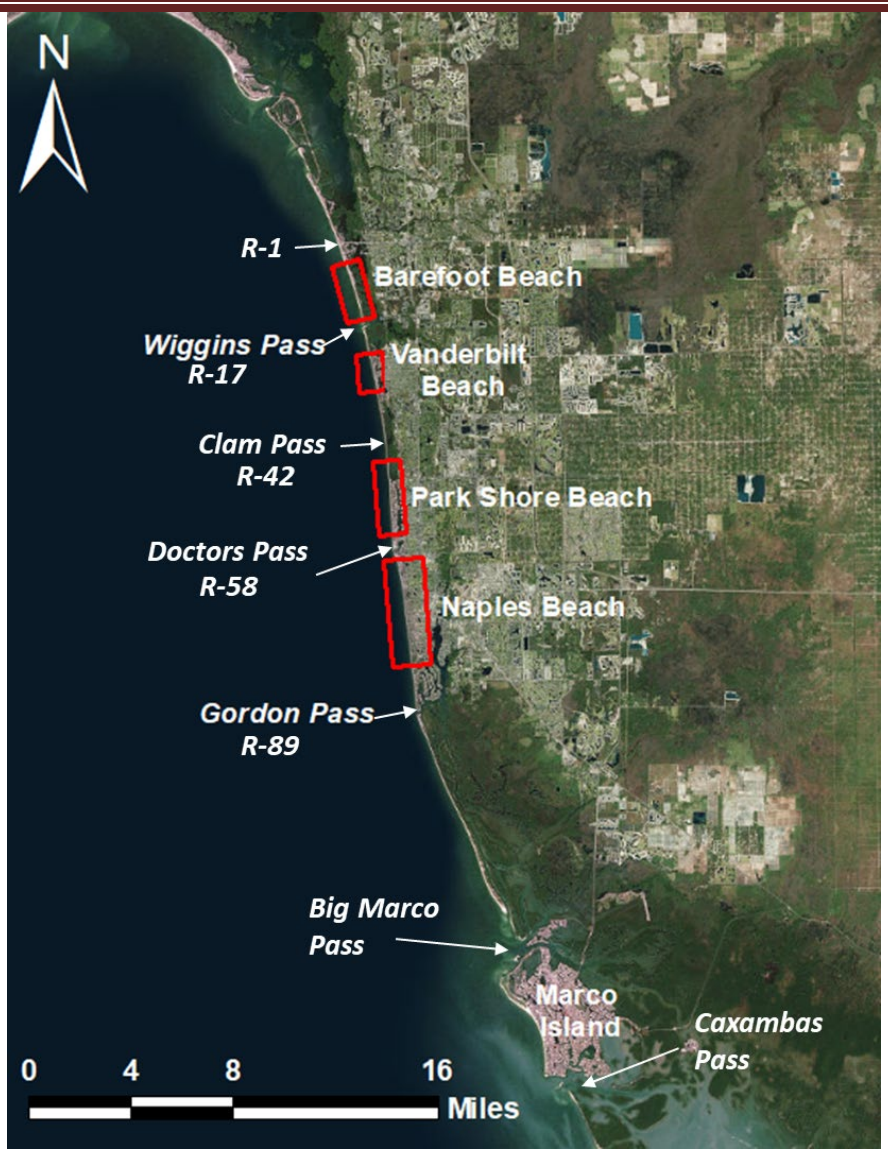


Figure 1.3. Collier North Beach and Inlet locations

1.2. SCOPE OF ENGINEERING APPENDIX AND ENGINEERING ANALYSIS

The Collier County CSRM Study will address potential structural and non-structural solutions in terms of mitigating the impacts of flooding and waves/erosion. This Engineering Appendix discusses the preliminary engineering and 10% design work conducted of the structural elements and measures of the Collier County CSRM Study. The preliminary design includes the evaluation of existing information and data, the evaluation of floodwalls, review and evaluation of beach nourishment, review and use of water levels from the updated FEMA Southwest Florida Study, evaluation of pump stations and drainage structures, and other structural elements and measures that would meet the objectives and goals of this study. This introduction to the Engineering Appendix provides a broad explanation behind both the preliminary engineering and 10% design work and incorporates reports as sub-appendices from Structural Engineering,

Geotechnical Engineering, Cost Estimates and Analysis, Hydraulics, Hydrology & Coastal Engineering and SBEACH support to the Beach-*fx* model.

1.3. ALTERNATIVES

1.3.1. FINAL ALTERNATIVES ARRAY

The Engineering Team members assisted the Planning and Environmental team members during the Plan Formulation process. An alternative plan is a set of one or more management measures functioning together to address one or more planning objectives. The team adopted a Cornerstone alternatives development approach where enhancing beach berm and dune features were included in every permutation of alternatives developments. Alternative A1, which was thought of as the *beaches only* alternative, was combined with various alternatives including structural and nonstructural measures for the inland bay areas. Based on the measures carried forward, both structural and nonstructural alternative plans were developed with more detailed analysis.

The final array was evaluated considering the following factors:

- Beach-fx – economic damages prevented
- G2CRM – economic damages prevented
- Health and Human Safety – Other Social Effects analysis
- Environmental – impact analysis such as water quality modeling and ESA considerations
- Construction – construction costs and feasibility
- Real Estate – acquisition costs and considerations
- Public Meetings – citizen input on focused array of alternatives

These categories were critical and considered the integral components of the four Principle and Guidelines criteria for evaluation of each alternative plan. The four criteria are completeness, efficiency, effectiveness, and acceptability. In addition, benefits were calculated using Beach-fx and G2CRM which are described in more detail in this Appendix and the Economics Appendix. The Final Array of Alternative in shown in Table 1.1. See the main report Chapter 6 and Plan Formulation Appendix for more detail on the formulation of Alternatives.

Table 1.1 Final Array of Alternatives.

Alternative	Components
A0 No Action	N/A
A1 (beach only)	Economically Justified Berm and Dune Measures from Gulf-facing Shoreline Analysis
A2 (beach + structural)	A1 + Structural
A3 (beach + nonstructural)	A1 + Nonstructural + Critical Infrastructure
A4 (combo)	Combination of structural and nonstructural measures from alternatives A1-A3

1.3.2. TENTATIVELY SELECTED PLAN

The tentatively selected plan (TSP) is Alternative A4 which is a combination of A1, A2, and A3 components. All components in the TSP are incrementally justified as separable elements. The TSP includes both structural and nonstructural measures to reduce the risk associated with future coastal storms in Collier County. The TSP is the outcome of the analyses which occurred to evaluate a variety of competing measures and alternative plans. Various design heights and associated water levels were investigated to determine the TSP scale and alignment. The TSP is a hybrid plan that includes structural measures, where they were found to be feasible and justified, and nonstructural measures elsewhere to provide greater coverage of the study area and further buy-down residual risk. The following Engineering Appendix and associated sub appendices details the engineering analysis undertaken to assist in the formulation of the TSP. See the main report Chapters 6 and 7, Plan Formulation Appendix, and Economics Appendix for more information regarding plan formulation and the TSP.

CHAPTER 2 EXISTING INFORMATION AND DATA

The first goal of this study was to analyze and review any existing data or information to assist with the analysis and in the 10% level of design. For the Collier County CSRM Study, a large amount of existing information, data and reports were readily available for this project. This Chapter will briefly discuss the existing data used for the engineering analysis.

2.1. SURVEY DATA/EXISTING ELEVATIONS

Bathymetric and topographic survey data was provided for the years 2013 – 2017. These surveys include beach nourishment post construction as well annual beach profile monitoring surveys collected by Collier County as part of their nourishment programs. The most recent monitoring survey was collected Post Hurricane Irma on September 25, 2017 - October 7, 2017. The survey contains beach profiles from FDEP monuments R – 17 through R-84, approximately Wiggins Pass to 5000 ft north of Gordon Pass and from R-139 to Caxambas Pass in Marco Island. Available surveys were also provided from the FDEP Coastal database from the years 1973 – Present which were used to help fill in areas not typically monitored by the County.

Hydrographic survey data exists for the years from 2016 - 2018 for the Federal channel that is maintained from Gordon Pass to Naples and from 2012 the channel between Naples and Big Marco Pass.

LiDAR was collected post Hurricane Irma between Marco Island and Barefoot Beach as part of emergency response efforts to assess elevations above and below water along the coast of Florida. LiDAR, which stands for Light Detection and Ranging, is an active form of remote sensing. LiDAR records laser pulses that strike an object and sends information back to a sensor. LiDAR measures the distance from a sensor to the object by determining the duration of time from the release of a laser pulse from the sensor to an object and the reflected laser pulse from the object to the LiDAR sensor. For terrestrial uses LiDAR uses near infrared laser (900–1064 nanometers) and water penetration green light (532 nanometers) for the water measurement. LiDAR of the ground (or bare earth) or water depths is called a digital elevation model (DEM). DEM contains z coordinates (elevation data) value along with x and y coordinates (position).

The DEM modeling grid from the FEMA Southwest Florida Study (see CHAPTER 4) combines several topographic and hydrographic data sets, which is described in more detail in the Hydraulics, Hydrology, & Coastal Sub Appendix. The modeling grid was used as an elevation source for bathymetric areas in the Collier County back bay and for topography outside the extents of the Post Irma LiDAR.

For all other hydrographic and topographic data which exists outside of these sources, data from the US Coastal Relief Model (CRM) for Florida and the East Gulf of Mexico was used and analyzed. NOAA's National Centers for Environmental Management (NCEI) U.S. Coastal Relief Model (CRM) contains grids or DEMS of the U.S. coastal zone which integrates offshore bathymetry with land topography from various data sources, including the U.S. National Ocean

Service Hydrographic Database, the U.S. Geological Survey, USACE, International Bathymetric Chart of the Caribbean Sea and the Gulf of Mexico, and academic institutions. CRM data varies in accuracy, with a vertical accuracy no better than 1 m any for and elevation values and a spatial resolution of 3 arc-seconds (or roughly 90 m) for the Florida and East Gulf of Mexico CRM (NGDC 2001). Due to its low resolution and accuracy, CRM data will only be utilized to fill in gaps in data where no more accurate dataset exists.

Analysis of the available survey data shows that the project area is low lying, with most areas showing an elevation less than 10 ft NAVD88, which is the approximate limits of the 100-yr (1%) flood extents from FEMA's 2012 Flood Insurance Study (FIS) limits. Figure 2.1 to Figure 2.3 show the existing elevations in Collier County from the DEM modeling grid from the FEMA Southwest Florida Study (Hydraulics, Hydrology, & Coastal Sub Appendix for details on the data used to create the grid). The color contour show all the areas at 10 ft NAVD88 and below. The 12 ft NAVD88 contour is also drawn on the figures.

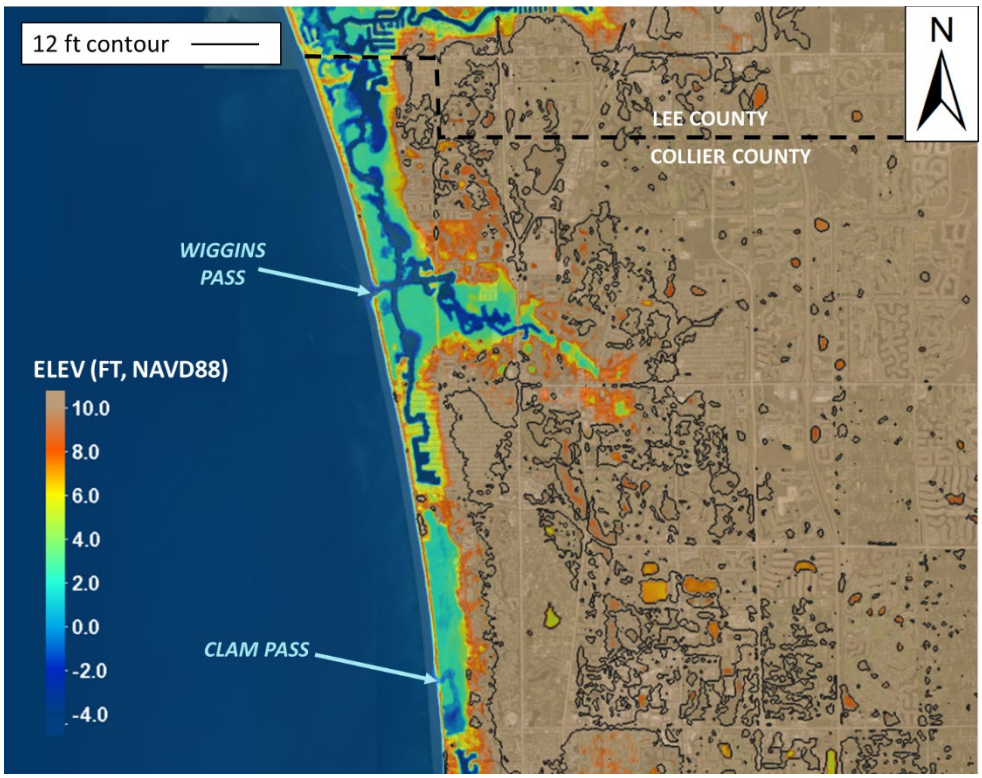


Figure 2.1 Existing elevations (North County Line to Clam Pass)

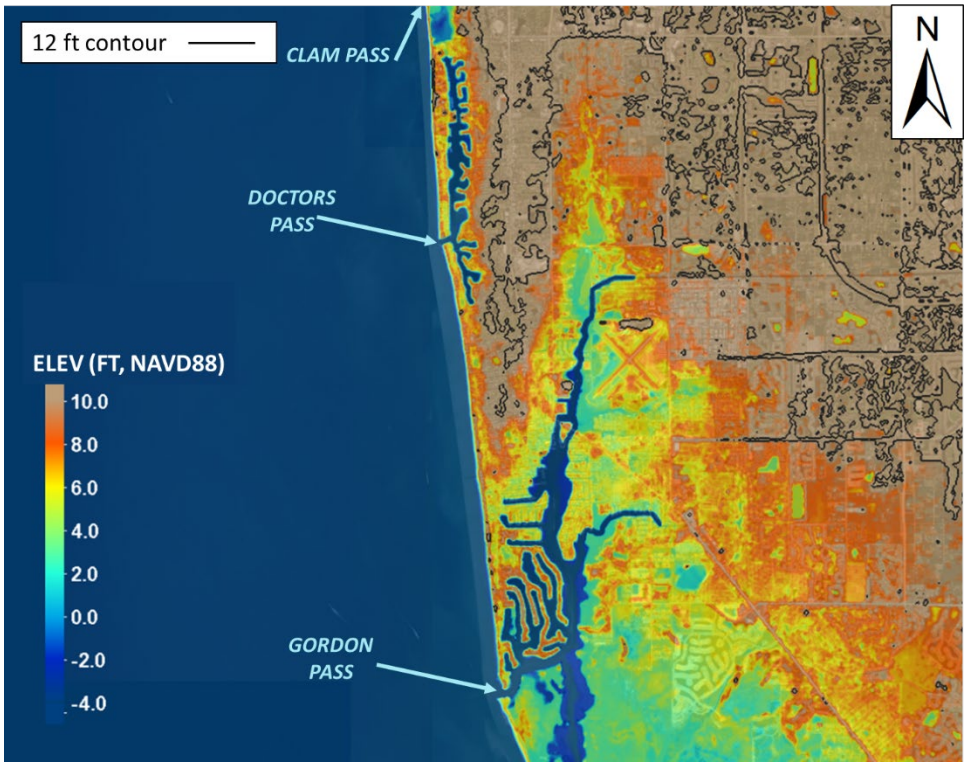


Figure 2.2 Existing elevations (Clam Pass to Gordon Pass)

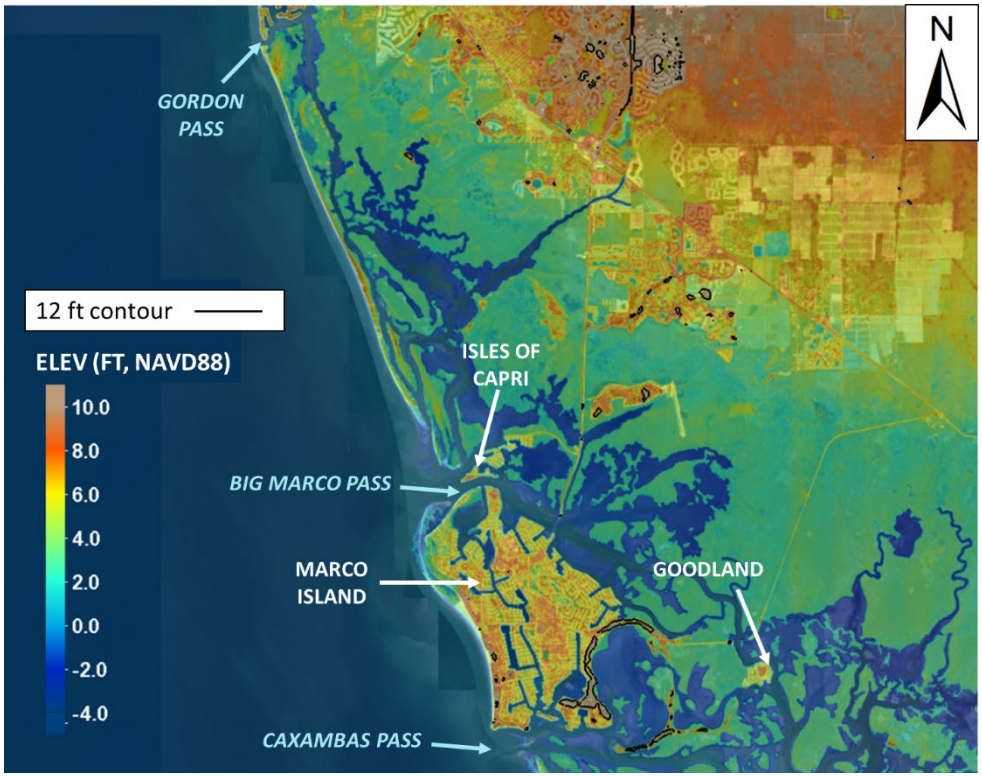


Figure 2.3 Existing elevations (Gordon Pass to Caxambas Pass)

2.2. SOIL BORINGS AND GEOTECHNICAL REPORTS

Existing geotechnical data used for the study consisted of nearby subsurface borings and geotechnical reports from following locations: the western end of Vanderbilt Beach Road, Wiggins Pass, The Naples Airport and north Tamiami Trail.

The data was used to generate a generalized subsurface profile for the study area. Typically the subsurface soils consisted of very loose to dense sand with some silt followed by weathered limestone and limestone as depth increases. Standard penetration testing (SPT) was typically conducted in the geotechnical reports as well as soil classifications, groundwater level measurements and permeability testing. For more information on geotechnical data see the Geotechnical Sub-Appendix.

2.3. NOAA NAPLES PIER TIDAL GAUGE RECORD

The records for the National Oceanic and Atmospheric Administration (NOAA) tidal Gauge at Naples Pier extend from March 1965 until the present and are considered representative of the tide history at and near the Collier County project areas. Further discussion on the Gauge and the information it provides can be read in the next chapter.

2.4. EXISTING COASTAL AND NAVIGATION PROJECTS

2.4.1. NORTH COUNTY

Historically, Collier County has maintained its beaches through the use of structures, beach nourishment, and inlet bypassing along the beaches in the North County from R-1 to R-89. Since 1996, through and existing non-Federal project, the County has nourished some portion of the beach every one to four years, placing approximately 3.25 million cubic yards in total in the North County. Sand sources have included offshore borrow areas (T1 is the source the County continues to lease from BOEM), inlet channels and ebb shoals, and truck hauls. The areas most frequently nourished by the County are Vanderbilt, Park Shore, and Naples beach. Prior to 1996, groins were commonly used to slow the littoral drift and hold sand. Part of the initial nourishment project in 1996 was the removal of 36 groins from the shoreline. Several groins still remain in the project area and have an influence on the adjacent beaches, forming fillets that change with the direction of wave approach (CPE 2011b).

Table 2.1 and Table 2.2 provide a summary of all the nourishment activities and approximate volumes since the construction of the initial major nourishment project completed in November 1995, Figure 2.5 provides an overview of the 2006 nourishment project nourishment areas compared to the layout of all the North County beaches to be considered in this study, and Figure 2.4 provides an example cross section of the fill template from a 2013 nourishment project.

Figure 2.5 shows that only Vanderbilt, Pelican Bay, Park Shore, and Naples beaches were

nourished in 2006, and these areas have been nourished most frequently by the County since the nourishment began in 1996. According to the County, Pelican Bay was included in 2006 as the community at Pelican Bay contributed funding to support beach nourishment in that location. There are several instances historically where the community of Pelican Bay contributed funds directly to include their beaches in a County beach nourishment project. If it were not for that funding, this section would not have been nourished.

Historically the beach template of the beach has included a "turtle friendly" stepped berm between 3 and 4 ft NAVD88 (Figure 2.4).

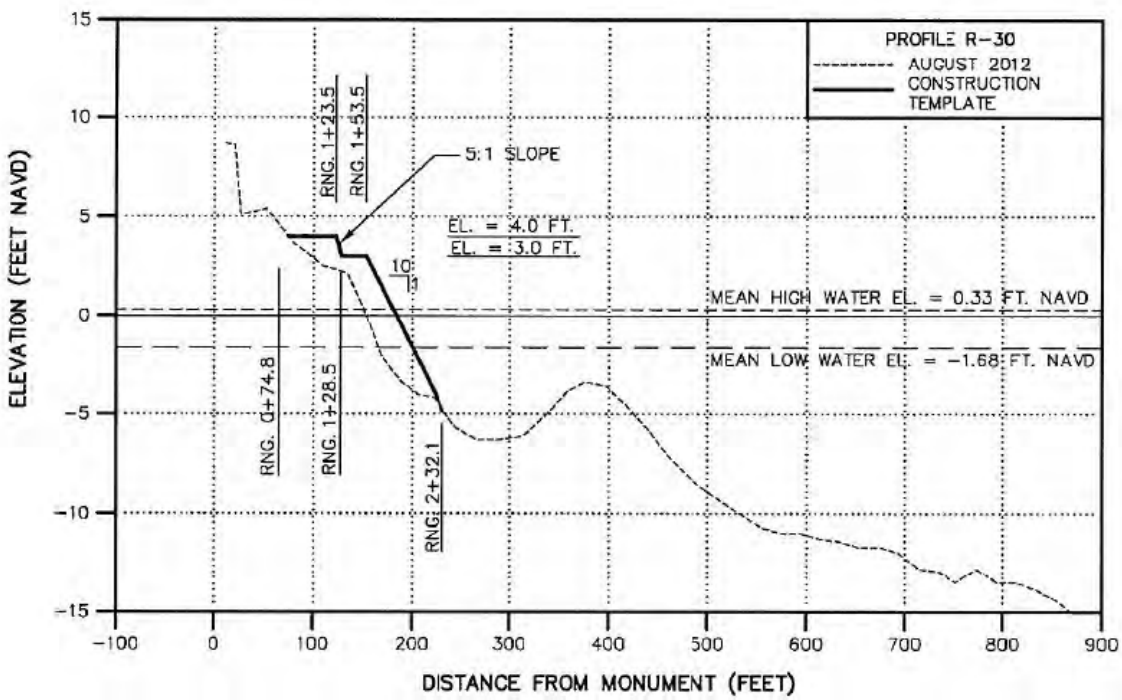


Figure 2.4 2013 Beach Template at R-30 (Vanderbilt Beach)

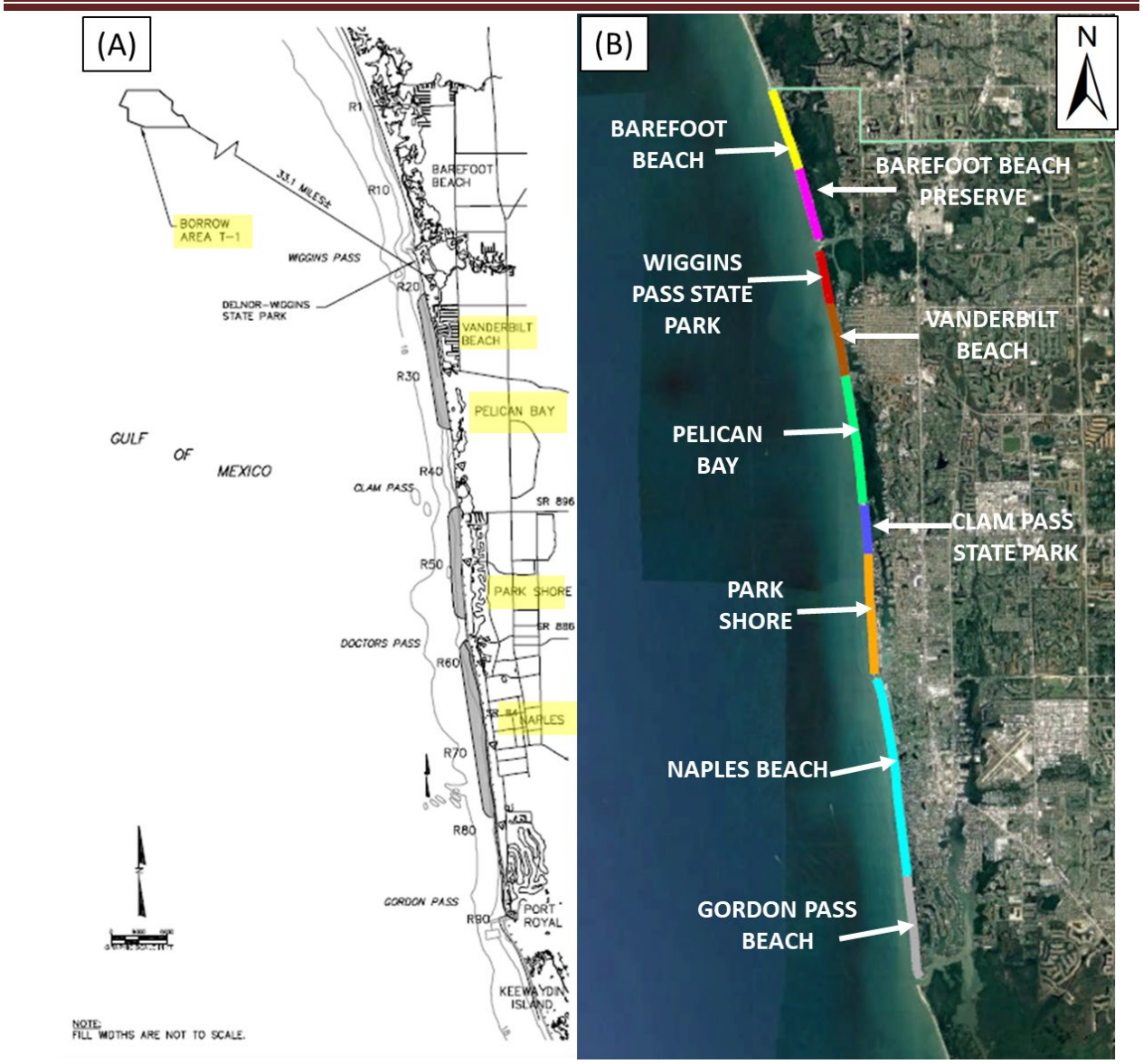


Figure 2.5 (A) 2006 North County Beach Renourishment Project Overview (B) Overview of all North County Beach areas

Table 2.1 Collier County Beach Nourishment History (R-1 to R-89 for 1996 thru 2019) (CPE, 2003), (H&M, 2018)

1996	Park Shore	R 50 to R-54	5,100	90,700	17.8	borrow areas and upland sand	Offshore Dredge
	Naples	R-58A to R-79	19,300	759,150	39.3		
Sep-96	Naples	Near Doctors, Likely 60-62	3000	55,000	18.3	Doctors Pass	Inlet Dredging-Beach Disposal
1996	Vanderbilt	R-22 to R-31	8,700	3,000	0.3	Upland	Truck Haul
1996 - 2002	Naples	R-58A to R-79	19,300	81,400	4.2	Upland	Truck Haul
2000-2002	Park Shore	R 50 to R-54	5,100	72,300	14.2	Upland	Truck Haul
2005	Barefoot Beach	R-11 to R-14	3050	68,500	22.5	Wiggins Pass	Inlet Dredging- Nearshore Disposal
2006	Vanderbilt	R-22 to R-30	9204	121,500	13.2	T1	Offshore Dredge
	Pelican Bay	R-31 to R-36	6104	56,950	9.3		
	Park Shore	R-45 to R-54	10549	141,750	13.4		
	Naples	R-58A to R-79	19427	347,420	17.9		
2009	Naples	R-60 to R-62	2097	32,500	15.5	Doctors Pass	Inlet Dredging- Nearshore Disposal
2010	Naples	R-58A to R-58	540	3,000	5.6	Upland	Truck Haul
2011	Park Shore	R-45 to R-46	2081	7,800	3.7	Upland	Truck Haul
	Naples	R-58A to R-58	1013	22,400	22.1		
2013	Barefoot Beach Preserve	R-12 to Inlet	4115	66,000	16.0	Wiggins Pass	Inlet Dredging- Nearshore Disposal
2013	South Pelican Bay	R-40 to Inlet	1012	10,400	10.3	Clam Pass	Inlet Dredging-Beach Disposal
	Clam Pass State Park	Inlet to R-44	2036	9,600	4.7		
2013	Naples	Inlet to R-60	3083	44,000	14.3	Doctors Pass	Inlet Dredging-Beach Disposal
2013	Vanderbilt	R-23 to R-30	7101	65,740	9.3	Upland	Truck Haul
	Pelican Bay	R-31 to R-36	4081	27,390	6.7		
	Park Shore	R-46 to R-55	7289	44,990	6.2		
	Clam Pass	R-43-R-45	3152	21,100	6.7		
2016	Clam Pass State Park	R-42 to R-43.5	1039	14,500	14.0	Clam Pass	Inlet Dredging-Beach Disposal
2016	Vanderbilt	R-24 to R-30	6141	35,470	5.8	Upland	Truck Haul
	Pelican Bay	R-34.6 to R-36.4	3054	8,500	2.8		
	Clam Pass	R-43-R-45	2416	15,600	6.5		
	Park Shore	R-46to R-53	4577	16,220	3.5		
2017	South Pelican Bay	R-40.5 to Inlet	1012	3,000	3.0	Clam Pass	Inlet Dredging-Beach Disposal
	Clam Pass State Park	Inlet to R-43	1039	2,500	2.4		
2018	Barefoot Beach Preserve	R-10 to Inlet	6162	53,396	8.7	Wiggins Pass	Inlet Dredging- Nearshore Disposal
	Wiggins Pass State Park	Inlet to R-21	5150	35,598	6.9		

Beach Placement in Collier Beach Project Area (Offshore Dredge):	23.5	1,840,270
Beach Placement in Collier Beach Project Area (Truck Haul):	5.4	698,210
Beach Placement in Collier Beach Project Area (Inlet Dredge with Beach Placement):	12.3	199,400
Beach Placement in Collier Beach Project Area (Inlet Dredge with Nearshore Placement):	14.7	508,620

Table 2.2 Collier County Beach Nourishment by Reach (CPE, 2003), (H&M, 2018)

	Offshore Dredging					Trucking					Inlet Dredging with Beach Disposal				
Beach Area	Total Volume (CY)	# Occurances	Interval From 1996 to present	Average CY per Occurance	AVG CY/LF	Total Volume (CY)	# Occurances	Interval From 1996 to present	Average CY per Occurance	AVG CY/LF	Total Volume (CY)	# Occurances	Interval From 1996 to present	Average CY per Occurance	AVG CY/LF
Vanderbilt	444,300	2	11.5	222,150	24.8	137,310	5	4.6	27,462	4.0					
Pelican Bay	56,950	1	23	56,950	9.3	35,890	2	11.5	17,945	5.0	13,400	2	11.5	6,700	6.6
Clam Pass State Park						70,516	3	7.7	23,505	7.2	47,200	5	4.6	9,440	9.2
Parkshore	232,450	2	11.5	116,225	14.9	217,494	5	4.6	43,499	3.0					
Naples	1,106,570	2	11.5	553,285	28.6	237,000	6	3.8	39,500	4.8	262,526	6	3.8	43,754	16.0

	Overall Summary	
Beach Area	Total Volume (CY)	AVG CY/LF
Vanderbilt	581,610	11.1
Pelican Bay	106,240	7.0
Clam Pass State Park	117,716	7.9
Parkshore	449,944	11.5
Naples	1,606,096	15.4

The information in the tables was primary used to assess the areas of highest priority to the County (Vanderbilt, Park Shore, and Naples) and helped guide how beach areas should be separated into model reaches for evaluation. The information, frequency of events and typical nourishments events, also helped to guide decisions made when including County existing nourishment practices in beach models as it is assumed if a federal project is not approved, the Sponsor will continue similar nourishing practices.

2.4.1.1. BORROW AREA CHARACTERISTICS

NATIVE BEACH SAND CHARACTERISTICS

The beach sands in the project area are gray fine grained sand with shell based on 2003 samples. The sands have been influenced by previous nourishment projects, truck haul sand, and bypassing at inlets. These activities have added moderate quantities of shell, minor rock, and coarse sand from upland sources.

Grain composites for the beach areas have been completed in the past and are shown in Table 2.3. These are composites of beach sand from areas that have been nourished in the past (Vanderbilt, Pelican Bay, Park Shore, and Naples from R–22 to R-79). The 2010 beach conditions assumes to have Borrow Area T1 Cut 1 characteristics as this is the borrow area utilized during the 2006 nourishment project (CPE 2011).

Table 2.3 Collier Beach Sand Composites (CPE 2011)

	Mean Grain Size (PHI)	Mean Grain Size (mm)	Sorting (PHI)	Silt (%)
1990 Native Beach	1.89	0.27	1.51	2.55
2003 Beach Composite	2.08	0.24	1.5	2.17
2010 Beach Composite	1.59	0.33	0.9	1.65

The 2010 Beach Composite was utilized for analysis of the North County beaches since inland trucking sources since 2010 have likely maintained a coarser sand composite because future offshore projects will continue to dredge sand from the Tom Hill (T1) site. Slightly coarser sand is also preferred as it could help mitigate encroachment on the nearshore hard bottom.

OFFSHORE SAND SOURCES

A letter was provided by the County (and attached as an Appendix) to the Norfolk District detailing their available sand sources for the next 50 years to cover their existing and estimated future nourishment requirements. These sources as summarized in this paragraph and the one following.

Off shore fill material is dredged from the ocean floor and then pumped or transported by barge to the re-nourishment area. Collier County's primary offshore borrow area is located in federal waters, originally named Borrow Area T1 and recently renamed to LE13-R109B. Collier County is the holder of the permits and BOEM lease for LE13-R109B and it is the intention of the County to maintain the lease until the source is exhausted. The permitted borrow area has 3.5 MCY of sediment, though it is possible to deepen LE13-R109B to gain more sediment and expand the southern boundary (LE13-R109A). Additionally, there is an adjacent hill sand source immediately west of T1, called T2 or LE14-R109 which has an estimated 4.9 MCY (Figure 2.6 and Figure 2.7) (APTIM, 2018). Considering these offshore sources there is unadjusted volume of approximately 13.4 MCY of offshore sand available. Applying a contingency reduction to the different areas to account for uncertainties in the quality of material in the areas, this may reduce to approximately 10 MCY. Collier County does have some potential additional sources of sand including inlet shoals and upland sand mines (discussed in the following section). By combining the existing permitted borrow area with potential expansion of T1, the inclusion of T2, along with sand from inlets and upland mines, the County is confident it has sediment the volume it needs in order to construct future beach nourishment projects over the next 50 years. Mean grain sizes of these borrow areas are provided in Table 2.4.

Table 2.4 North County Offshore Sand Source Characteristics (CPE 2011, APTIM 2018)

	Mean Grain Size (mm)	Silt (%)
Toms Hill I (T1) LE13-R109B	0.32	1.75
T1 (Expanded) LE13-R109A	0.25 – 35	Unknown
T2 LE14 – R109	0.25 - 35	Unknown

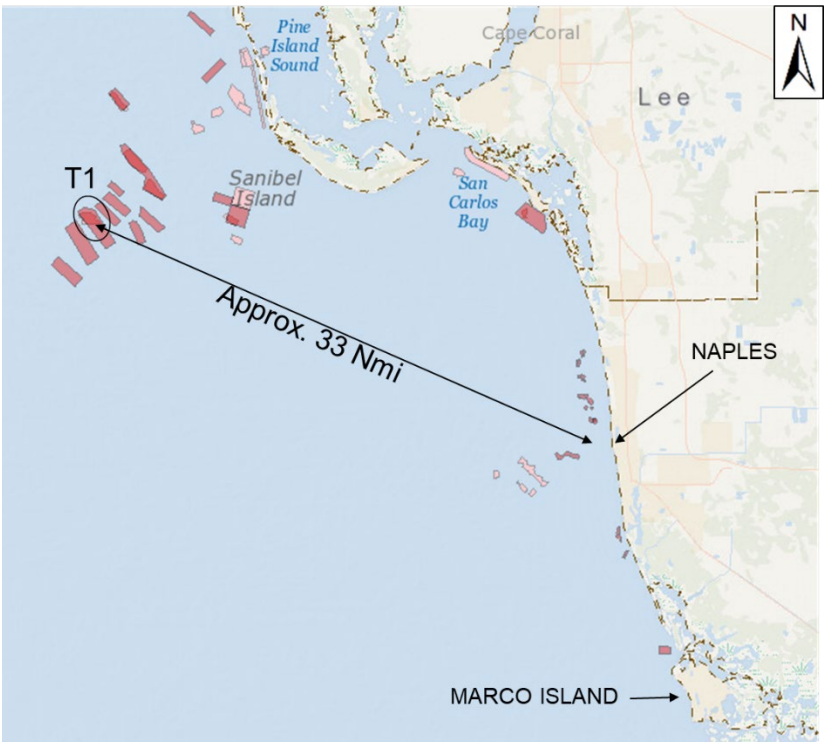


Figure 2.6 Location of Offshore Sand Sources

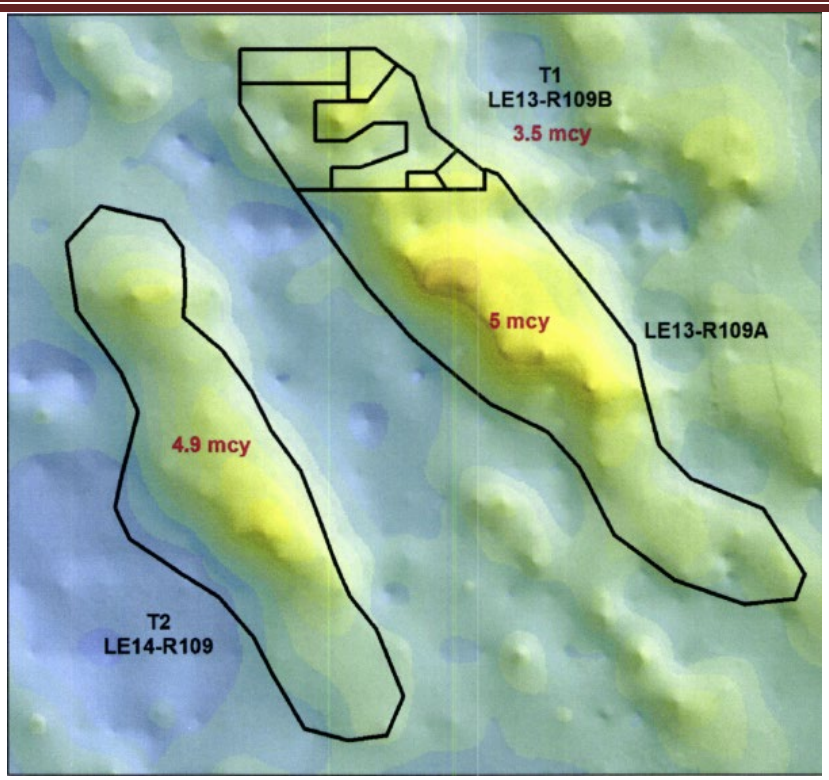


Figure 2.7 Color-shaded relief image of bathymetry surrounding Tom's Hills. Volumes are labeled in red.

INLET SAND SOURCES

The County has also used inlets throughout the North County (Wiggins Pass, Clam Pass, and Doctors Pass) to supplement the primary borrow areas and to address hot spots.

Table 2.5 North County Inlet Sand Source Characteristics (APTIM 2018, Turrell 2014)

	Mean Grain Size (mm)	Silt (%)
Wiggins Pass	0.36-0.52	1.7-2.6
Doctors Pass	0.18	0.9
Clam Pass	0.2	Unknown

INLAND SAND SOURCES

Collier County has utilized inland sand sources successfully to address hot spots along the County beaches. Inland sources excavate fill material from local quarries or mines within the area. The fill material is then transported directly to the re-nourishment area. The Immokalee Mines in northeast Collier County have been utilized most frequently in the past. Other Possible inland sources include:

- Witherspoon Mine
- Ortana Mine

- Garcia Sand Mine
- SDI (Atlantic Civil, Inc.) Mine

2.4.2. MARCO ISLAND

The Marco Island beaches have also been maintained periodically since an initial nourishment and erosion control project in 1991 using sand from borrow areas within the Big Marco, Capri Pass, and Caxambas Pass ebb shoals. This project nourished 2.64 miles of beach from R-135t o R-139 and from R-143 to R-148. Since the 1991 nourishment, the northern beach reach from R-135.5 to R-143 tends to accrete while the beach segment between R-143 and R-148 on the southern end of Marco Island is designated as critically eroded. Recent projects in South Marco Island, approximately ranging from DEP reference monument from R-143 to Caxambas Pass, have nourished the beach using sand dredged from Caxambas Pass and truck hauls. Table 2.7 provides a summary of all the nourishment activities along Marco Island and approximate volumes since 1991 and Figure 2.8 provides an overview of the nourishment project layout in 2013. The native beach mean grain size for Marco Island is 0.26 mm with a mean grain range of 0.17 to 0.43 mm. Caxambas Pass, which is frequently utilized to nourish South Marco Island is slightly coarser with a mean grain size of 0.35 mm and grain size range of 0.19 to 0.68. Big Marco Pass to the north which could be used as a sand sources for Marco Island is slightly finer with a mean grain size of 0.21 and grain size range of 0.12 to 0.52 mm (CEC 2011). The native beach grain size was used for all analysis of Marco Island beaches.

Table 2.6 Marco Island Native and Borrow Area Sediment Characteristics (APTIM 2018, CEC 2011)

	Mean Grain Size (mm)	Mean Grain Size Range (mm)	Silt (%)
Native Beach	0.26	0.17 to 0.43	< 5%
Caxambas Pass	0.35	0.19 to 0.68	2.93
Big Marco Pass	0.21	0.12 to 0.52	2.04

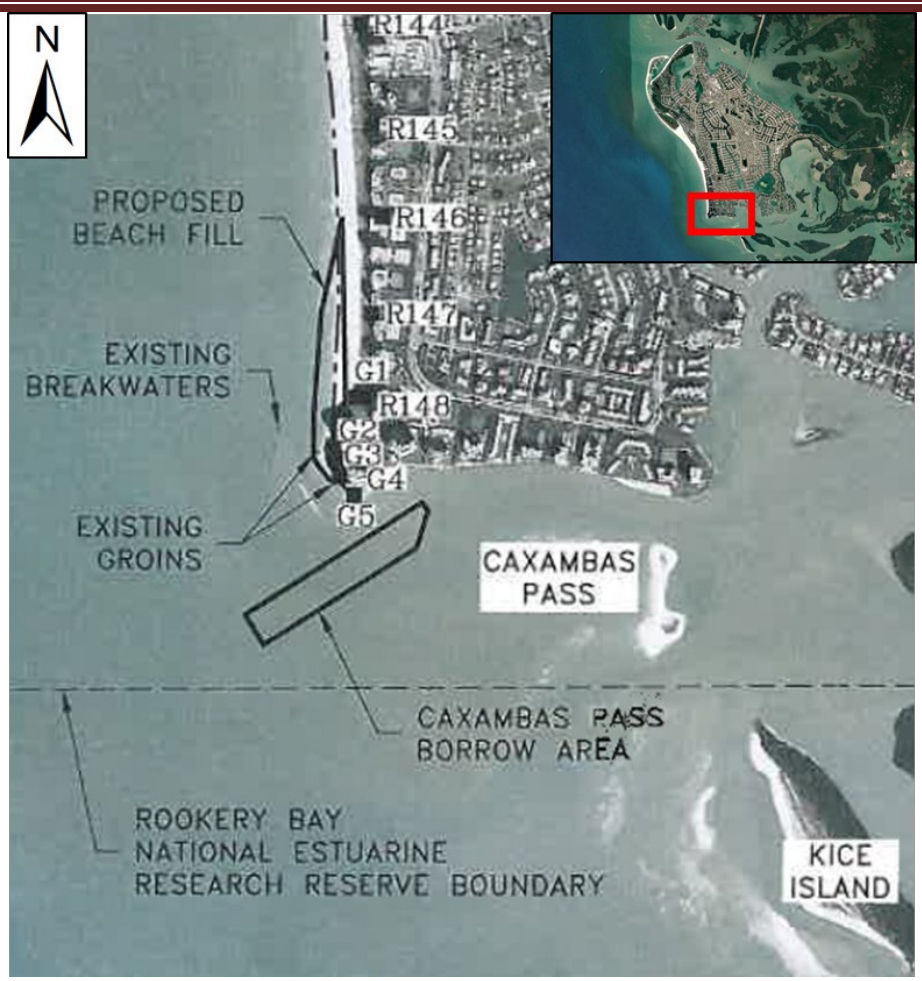


Figure 2.8 2013 South Marco Island Beach Renourishment Project Overview

Table 2.7 Marco Island Nourishment History for 1991 thru 2019 (H&M, 2018b)

Date	Location	Amount [CY]	BA	Method
1991	South Marco Island	575,000		
1997	South Marco Island	60,000	Caxambas Pass Inlet	Dredging
1998 & 1999	South Marco Island	10,000	Upland	Trucking
2007 Jan	South Marco Island R-144 South to G-2	176,000	Caxambas Pass Inlet	Dredging
2013 Apr	South Marco Island R-146 to G-4	80,000	Caxambas Pass Inlet	Dredging
2016 Nov	R-144 south to Station G-2	13,200	Upland Stewart Mining (Immokalee, FL)	Trucking
2019	R-135.5 south to R-141.5	North Beach Regrading	N/A	

2.5. PREVIOUS STUDIES AND REPORTS

Collier County has previously conducted several engineering analyses and monitoring reports on its existing projects. The reports from these studies were reviewed to better understand existing coastal management measures taken by the County and see what information and data from these studies could be used in the CSRM Study. The previous study reports provided by Collier County are mentioned in the main report and several are included in the references.

CHAPTER 3 PHYSICAL CONDITIONS

3.1. CLIMATE

Collier County's climate is Tropical Savanna with the temperature averaging 75°F annually. Winters are generally warm and dry, and summers are hot and dry. The average annual precipitation is approximately 53.45 inches with average monthly amounts ranging from 1.5 inches in December to 8.5 inches in June. The rainy season, from May to October, coincides with hurricane season, during which Collier County receives 80 percent of its annual rainfall (FEMA 2012). Collier County has a considerable history of hurricane and tropical storm events impacting the area. At least 74 tropical cyclones have passed within 75 nautical miles of Naples since 1851, one storm nearly every 2.2 years. Of those, 30 have been hurricanes, once hurricane every 5.4 years. Some notable hurricanes include Donna (1960), Hurricane Andrew (1992), Hurricane Georges (1998), and Hurricane Irma (2017).

3.2. HORIZONTAL AND VERTICAL DATUMS

The horizontal datum for this study is tied to the State Plane Coordinate System using North American Datum of 1983 (NAD83, Florida East, 901). Distances are in feet by horizontal measurement. Coordinates are Florida East Zone. The vertical datum for this study is tied to the North American Vertical Datum of 1988 (NAVD88), a requirement of ER 1110-2-8160. Elevations are in feet.

3.3. WINDS & WAVES

The information provided in this sections helps to provide visualizations of the historical wind and wave climate along the Collier County shoreline. Please see CHAPTER 4 FEMA SOUTHWEST FLORIDA (SWFL) STUDY and the HYDRAULICS, HYDROLOGY & COASTAL SUB-APPENDIX for information on the water level and wave data used for the Engineering Evaluation.

The wind and wave directions were analyzed at two WIS stations, one closer to shore in shallower water and another in deeper water (Figure 3.1). Hindcast data was available for each of these stations from the years 1980 – 2014.

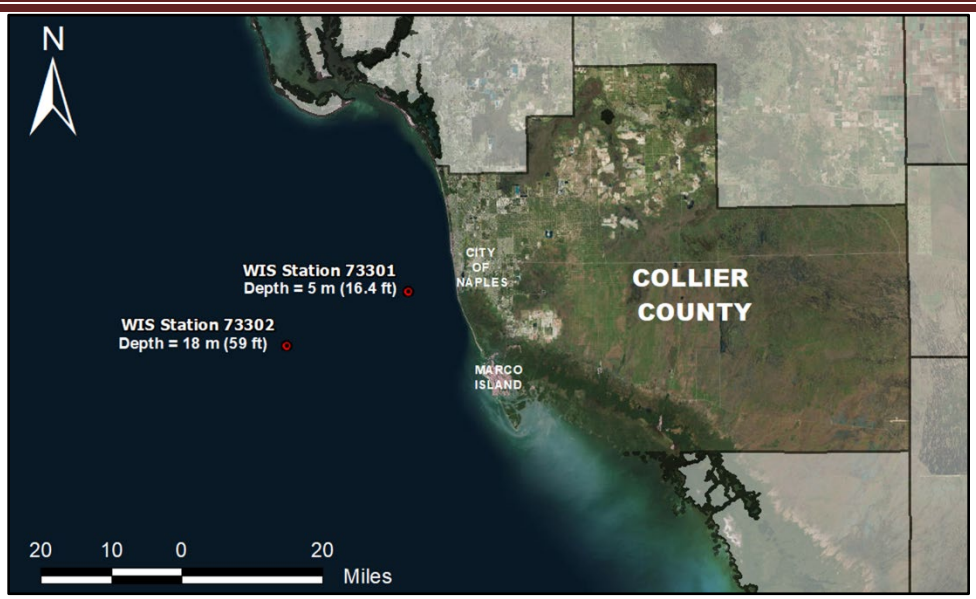
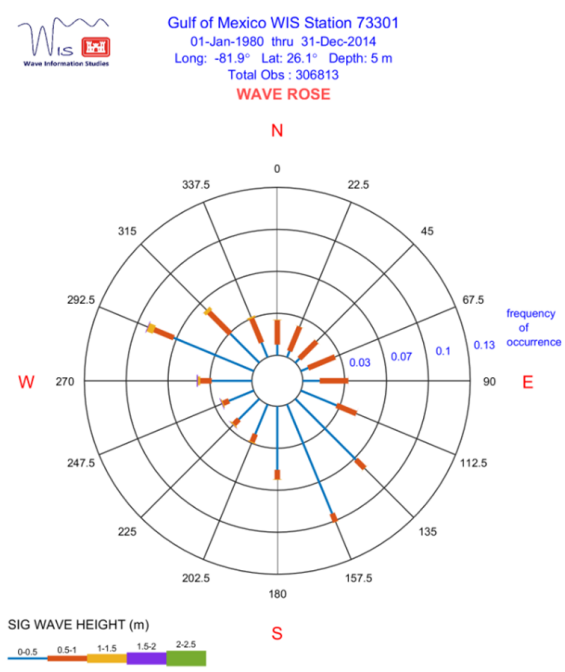


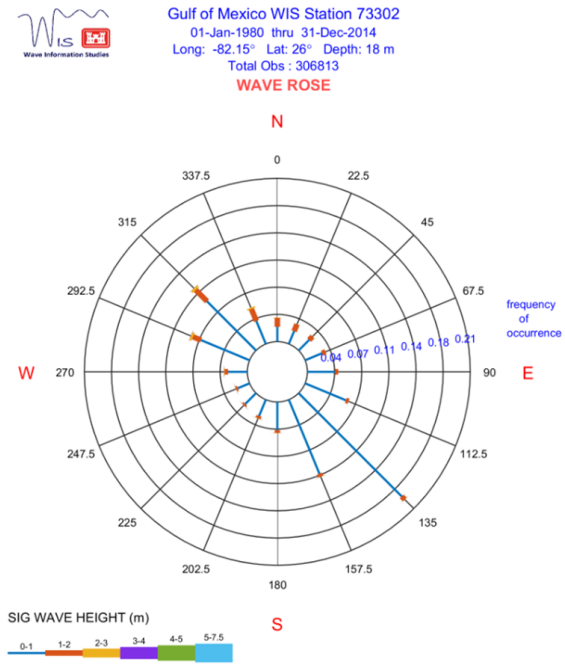
Figure 3.1. WIS Stations locations

For both stations, the prevailing wave directions impacting Collier County are those from southeast and northwest. The waves coming from the northerly direction tend to be higher than the waves from the southern direction. Figure 3.2 and Figure 3.3 display plots wave roses and summaries of wave conditions off the coast of the City of Naples for shallow water and deep water conditions respectively.



WAVE DIR	% of Total	Mean Hmo (ft)	Max Hmo (ft)	Mean Tp (sec)
N	2.8%	1.97	6.56	3.2
NNE	2.6%	1.97	6.56	3.1
NE	2.4%	1.97	6.23	3.0
ENE	2.9%	1.64	6.56	3.0
E	3.6%	1.64	6.23	3.0
ESE	4.7%	1.64	6.23	2.8
SE	7.8%	0.98	6.89	2.8
SSE	11.5%	0.66	6.56	2.9
S	6.0%	0.98	6.23	3.2
SSW	3.3%	1.31	5.90	4.0
SW	3.0%	1.31	5.90	4.5
WSW	2.8%	1.31	5.90	4.9
W	4.5%	1.31	5.90	5.4
WNW	9.5%	1.31	6.23	5.2
NW	6.2%	1.64	6.23	4.4
NNW	3.4%	1.97	5.58	3.5
	77.0%	1.31	6.89	3.70
% Calms (< 0.1 m)	23.1%			

Figure 3.2. Shallow Water Wave Rose and Significant Wave Height Summary (from WIS Data).



WAVE DIR	% of Total	Mean Hmo (ft)	Max Hmo (ft)	Mean Tp (sec)
N	3.1%	2.95	12.46	4.3
NNE	2.8%	2.95	11.48	4.0
NE	2.6%	2.62	14.43	3.8
ENE	2.9%	2.62	11.15	3.7
E	4.0%	2.30	13.78	3.7
ESE	6.1%	1.97	12.79	3.8
SE	19.6%	1.31	12.79	3.6
SSE	10.9%	1.31	20.99	3.6
S	4.1%	1.64	21.65	4.0
SSW	2.7%	1.97	13.78	4.7
SW	2.3%	1.97	13.45	4.9
WSW	2.0%	1.97	14.76	5.3
W	3.1%	1.97	15.09	5.4
WNW	8.4%	1.97	14.76	5.5
NW	11.6%	1.97	16.40	5.1
NNW	5.2%	2.95	9.84	5.1
	91.4%	1.97	21.65	4.30
% Calms (< 0.1 m)	8.7%			

Figure 3.3. Deep Water Wave Rose and Significant Wave Height Summary (from WIS Data).

Table 3.1- Table 3.4 breakdown the wave occurrences for ranges of significant wave heights, wave periods, and directions.

Table 3.1 Shallow water percent occurrence of significant wave height and direction (from WIS data)

GULF OF MEXICO STATION 73301 (1980 – 2014)

	PERCENT OCCURRENCE OF SIGNIFICANT WAVE HEIGHT AND DIRECTION				
	METERS				
WIND DIR	0.1 - 0.49	0.5 - 0.99	1 - 1.49	1.5 +	TOTAL
	FEET				
	0.3 - 1.6	1.6 - 3.2	3.3 - 4.9	4.9 +	
N	0.75%	1.98%	0.04%	0.00%	2.8%
NNE	0.43%	2.12%	0.02%	0.00%	2.6%
NE	0.37%	1.96%	0.01%	0.00%	2.3%
ENE	0.50%	2.37%	0.01%	0.00%	2.9%
E	1.14%	2.41%	0.01%	0.00%	3.6%
ESE	2.73%	1.94%	0.01%	0.00%	4.7%
SE	6.72%	1.10%	0.02%	0.00%	7.8%
SSE	10.72%	0.77%	0.01%	0.00%	11.5%
S	5.20%	0.79%	0.04%	0.00%	6.0%
SSW	2.54%	0.69%	0.08%	0.01%	3.3%
SW	2.30%	0.55%	0.09%	0.03%	3.0%
WSW	2.14%	0.51%	0.09%	0.04%	2.8%
W	3.26%	0.91%	0.19%	0.08%	4.4%
WNW	7.20%	1.66%	0.53%	0.05%	9.4%
NW	3.51%	2.40%	0.31%	0.01%	6.2%
NNW	1.14%	2.09%	0.15%	0.00%	3.4%
TOTAL	50.64%	24.24%	1.59%	0.23%	76.7%
			PERCENT OF CALMS (< 0.1 m)		23.1%

Table 3.2. Shallow water percent occurrence of spectral wave period (from WIS data)

GULF OF MEXICO STATION 73301 (1980 – 2014)

	PERCENT OCCURRENCE OF SPECTRAL WAVE PERIOD (IN SECONDS)										
WIND DIR	<3.0	3.0 - 3.9	4.0 - 4.9	5.0 - 5.9	6.0 - 6.9	7.0 - 7.9	8.0 - 8.9	9.0 - 9.9	10.0 - 11.9	12.0 - LONGER	TOTAL
N	0.99%	1.72%	0.05%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	2.8%
NNE	1.01%	1.56%	0.01%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	2.6%
NE	1.24%	1.11%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	2.3%
ENE	1.85%	1.02%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	2.9%
E	2.84%	0.71%	0.01%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	3.6%
ESE	4.00%	0.67%	0.01%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	4.7%
SE	6.77%	1.04%	0.02%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	7.8%
SSE	9.40%	2.03%	0.06%	0.00%	0.00%	0.00%	0.00%	0.01%	0.00%	0.00%	11.5%
S	2.98%	2.59%	0.32%	0.04%	0.04%	0.02%	0.01%	0.03%	0.00%	0.00%	6.0%
SSW	0.79%	1.36%	0.65%	0.25%	0.11%	0.06%	0.04%	0.05%	0.01%	0.00%	3.3%
SW	0.48%	0.87%	0.82%	0.42%	0.19%	0.07%	0.04%	0.08%	0.01%	0.00%	3.0%
WSW	0.28%	0.77%	0.80%	0.38%	0.19%	0.11%	0.06%	0.17%	0.01%	0.00%	2.8%
W	0.26%	1.00%	1.16%	0.67%	0.46%	0.28%	0.24%	0.39%	0.01%	0.00%	4.4%
WNW	0.49%	2.21%	2.83%	1.44%	0.87%	0.54%	0.47%	0.60%	0.00%	0.00%	9.4%
NW	1.07%	2.38%	1.43%	0.37%	0.26%	0.23%	0.25%	0.23%	0.00%	0.00%	6.2%
NNW	1.06%	1.89%	0.32%	0.01%	0.01%	0.02%	0.04%	0.05%	0.00%	0.00%	3.4%
	35.5%	22.9%	8.5%	3.6%	2.1%	1.3%	1.1%	1.6%	0.0%	0.0%	76.7%
										PERCENT OF CALMS (< 0.1 m)	23.1%

Table 3.3. Deep water percent occurrence of significant wave height and direction (from WIS data)

GULF OF MEXICO STATION 73302 (1980 – 2014)

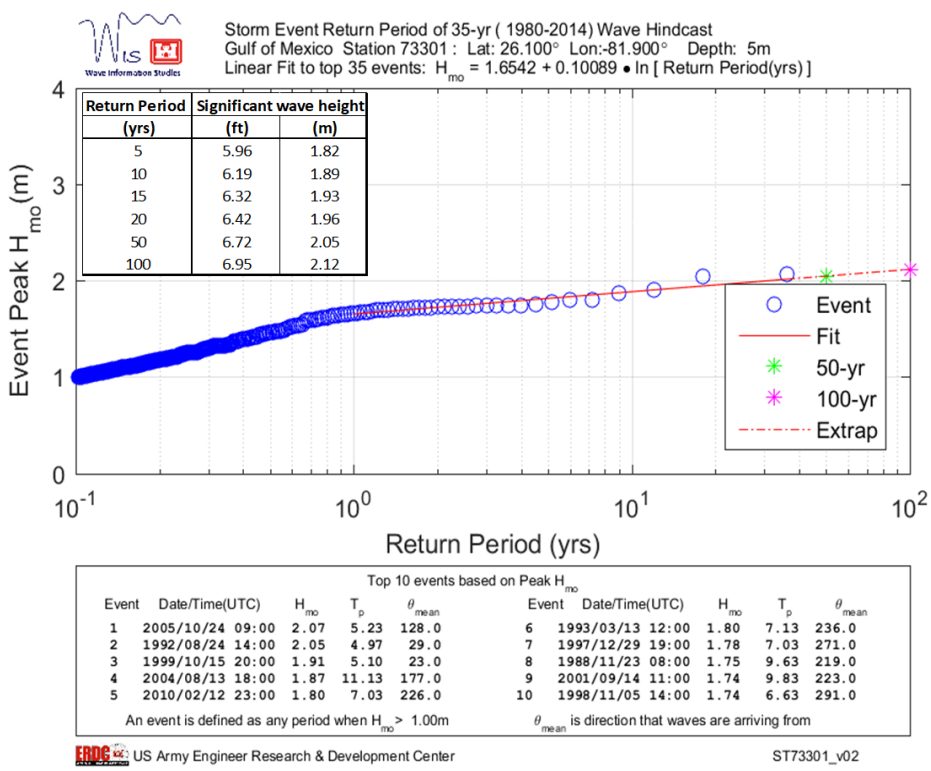
	PERCENT OCCURRENCE OF SIGNIFICANT WAVE HEIGHT AND DIRECTION								
	METERS								
WIND DIR	0.1 - 0.49	0.5 - 0.99	1 - 1.49	1.5 - 1.99	2 - 2.49	2.5 - 2.99	3 - 3.49	3.5 +	TOTAL
	FEET								
	0.3 - 1.6	1.6 - 3.2	3.3 - 4.9	4.9 - 6.5	6.6 - 8.2	8.2 - 9.8	9.8 - 11.4	11.5 +	
N	0.44%	1.45%	0.92%	0.26%	0.04%	0.00%	0.00%	0.00%	3.1%
NNE	0.41%	1.35%	0.85%	0.14%	0.01%	0.00%	0.00%	0.00%	2.8%
NE	0.44%	1.46%	0.56%	0.07%	0.02%	0.00%	0.00%	0.00%	2.6%
ENE	0.53%	1.82%	0.45%	0.04%	0.01%	0.00%	0.00%	0.00%	2.9%
E	0.86%	2.70%	0.41%	0.03%	0.00%	0.00%	0.00%	0.00%	4.0%
ESE	2.00%	3.70%	0.34%	0.03%	0.01%	0.00%	0.00%	0.00%	6.1%
SE	14.64%	4.39%	0.53%	0.05%	0.02%	0.01%	0.00%	0.00%	19.6%
SSE	8.43%	2.00%	0.36%	0.06%	0.02%	0.02%	0.01%	0.00%	10.9%
S	2.60%	1.05%	0.28%	0.09%	0.02%	0.01%	0.00%	0.00%	4.1%
SSW	1.43%	0.85%	0.24%	0.09%	0.03%	0.02%	0.00%	0.00%	2.7%
SW	1.28%	0.70%	0.17%	0.06%	0.02%	0.01%	0.00%	0.01%	2.3%
WSW	1.04%	0.61%	0.17%	0.05%	0.03%	0.01%	0.01%	0.01%	1.9%
W	1.72%	0.81%	0.34%	0.12%	0.04%	0.03%	0.01%	0.00%	3.1%
WNW	4.93%	2.03%	0.72%	0.41%	0.21%	0.08%	0.02%	0.00%	8.4%
NW	6.61%	2.54%	1.34%	0.76%	0.25%	0.09%	0.01%	0.00%	11.6%
NNW	1.43%	1.81%	1.18%	0.52%	0.15%	0.04%	0.00%	0.00%	5.1%
Total	48.76%	29.26%	8.87%	2.78%	0.89%	0.33%	0.08%	0.03%	91.0%
								PERCENT OF CALMS (< 0.1 m)	8.7%

Table 3.4. Deep water percent occurrence of spectral wave period (from WIS data)

GULF OF MEXICO STATION 73302 (1980 – 2014)

	PERCENT OCCURRENCE OF SPECTRAL WAVE PERIOD (IN SECONDS)										
WIND DIR	<3.0	3.0 - 3.9	4.0 - 4.9	5.0 - 5.9	6.0 - 6.9	7.0 - 7.9	8.0 - 8.9	9.0 - 9.9	10.0 - 11.9	12.0 - LONGER	TOTAL
N	0.19%	1.03%	1.45%	0.32%	0.08%	0.03%	0.01%	0.00%	0.00%	0.00%	3.1%
NNE	0.23%	1.20%	1.23%	0.08%	0.01%	0.00%	0.00%	0.00%	0.00%	0.00%	2.8%
NE	0.28%	1.36%	0.86%	0.06%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	2.6%
ENE	0.33%	1.73%	0.77%	0.03%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	2.9%
E	0.41%	2.60%	0.96%	0.03%	0.01%	0.00%	0.00%	0.00%	0.00%	0.00%	4.0%
ESE	0.53%	3.54%	1.95%	0.06%	0.01%	0.00%	0.00%	0.00%	0.00%	0.00%	6.1%
SE	5.34%	9.19%	4.82%	0.24%	0.03%	0.01%	0.00%	0.00%	0.00%	0.00%	19.6%
SSE	2.91%	5.42%	2.18%	0.29%	0.06%	0.02%	0.00%	0.01%	0.00%	0.00%	10.9%
S	0.52%	1.89%	1.13%	0.32%	0.10%	0.03%	0.02%	0.04%	0.00%	0.00%	4.1%
SSW	0.16%	0.76%	0.86%	0.56%	0.20%	0.07%	0.02%	0.05%	0.00%	0.00%	2.7%
SW	0.12%	0.50%	0.78%	0.50%	0.20%	0.05%	0.03%	0.07%	0.00%	0.00%	2.3%
WSW	0.07%	0.37%	0.66%	0.36%	0.19%	0.09%	0.06%	0.13%	0.00%	0.00%	1.9%
W	0.09%	0.47%	0.90%	0.76%	0.38%	0.18%	0.14%	0.17%	0.00%	0.00%	3.1%
WNW	0.11%	1.43%	2.46%	1.83%	1.12%	0.63%	0.45%	0.37%	0.00%	0.00%	8.4%
NW	0.24%	2.44%	3.84%	2.27%	1.44%	0.74%	0.42%	0.21%	0.00%	0.00%	11.6%
NNW	0.16%	0.87%	1.63%	1.34%	0.69%	0.27%	0.14%	0.02%	0.00%	0.00%	5.1%
Total	11.7%	34.8%	26.5%	9.1%	4.5%	2.1%	1.3%	1.1%	0.0%	0.0%	91.0%
										PERCENT OF CALMS (< 0.1 m)	8.7%

The statistical summary of extreme wave heights for both stations are shown in Figure 3.4 and Figure 3.5.



Return Period (yrs)	Significant wave height (ft)	(m)
5	5.96	1.82
10	6.19	1.89
15	6.32	1.93
20	6.42	1.96
50	6.72	2.05
100	6.95	2.12

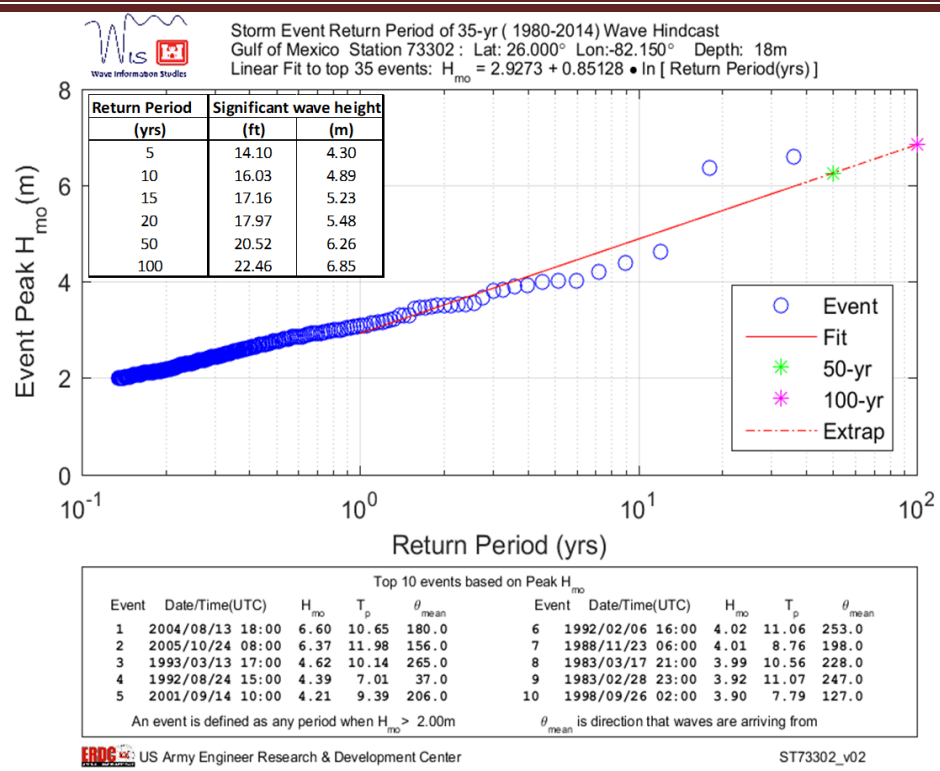
Top 10 events based on Peak H_{mo}

Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}	Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}
1	2005/10/24 09:00	2.07	5.23	128.0	6	1993/03/13 12:00	1.80	7.13	236.0
2	1992/08/24 14:00	2.05	4.97	29.0	7	1997/12/29 19:00	1.78	7.03	271.0
3	1999/10/15 20:00	1.91	5.10	23.0	8	1988/11/23 08:00	1.75	9.63	219.0
4	2004/08/13 18:00	1.87	11.13	177.0	9	2001/09/14 11:00	1.74	9.83	223.0
5	2010/02/12 23:00	1.80	7.03	226.0	10	1998/11/05 14:00	1.74	6.63	291.0

An event is defined as any period when H_{mo} > 1.00m

θ_{mean} is direction that waves are arriving from

Figure 3.4. Shallow water storm event return period (from WIS data)



Return Period (yrs)	Significant wave height (ft)	(m)
5	14.10	4.30
10	16.03	4.89
15	17.16	5.23
20	17.97	5.48
50	20.52	6.26
100	22.46	6.85

Top 10 events based on Peak H_{mo}

Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}	Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}
1	2004/08/13 18:00	6.60	10.65	180.0	6	1992/02/06 16:00	4.02	11.06	253.0
2	2005/10/24 08:00	6.37	11.98	156.0	7	1988/11/23 06:00	4.01	8.76	198.0
3	1993/03/13 17:00	4.62	10.14	265.0	8	1983/03/17 21:00	3.99	10.56	228.0
4	1992/08/24 15:00	4.39	7.01	37.0	9	1983/02/28 23:00	3.92	11.07	247.0
5	2001/09/14 10:00	4.21	9.39	206.0	10	1998/09/26 02:00	3.90	7.79	127.0

An event is defined as any period when H_{mo} > 2.00m

θ_{mean} is direction that waves are arriving from

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ST73302_v02

Figure 3.5 Deep water storm event return period (from WIS data)

Figure 3.6 and Figure 3.7 display plots winds roses off the coast of the City of Naples for shallow water and deep water conditions respectively and Table 3.5 and Table 3.6 display the percent occurrence of the wind speeds and directions. Dominant wind directions are from the ENE, E, and ESE.

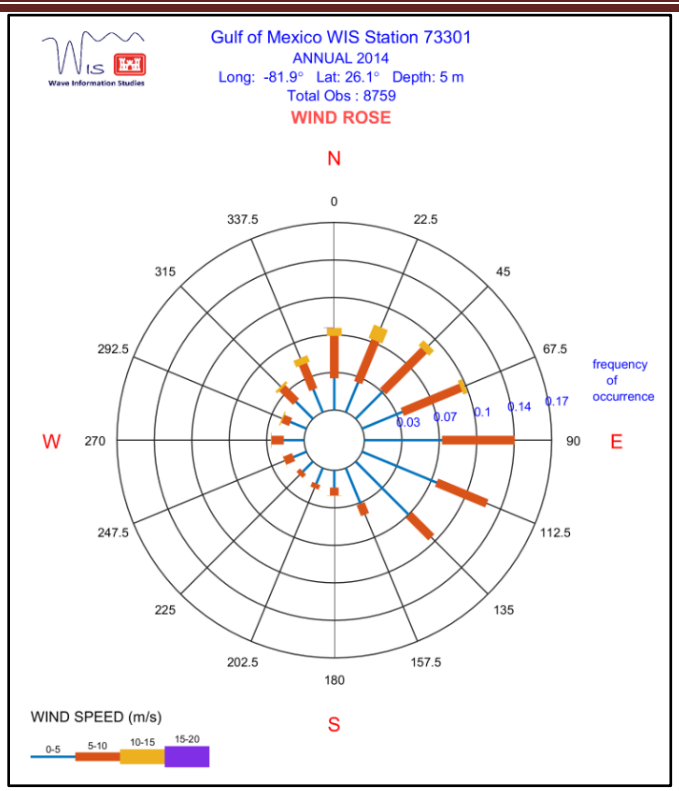


Figure 3.6. Image of Shallow Water Wind Rose (from WIS Data).

Table 3.5. Shallow water percent occurrence of wind speed and direction (from WIS data)

WIND DIR	PERCENT OCCURRENCE OF WIND SPEED AND DIRECTION (M/S)										TOTAL
	<2.5	2.5 - 4.9	5.0 - 7.4	7.5 - 9.9	10.0 - 12.4	12.5 - 14.9	15.0 - 17.4	17.5 - 19.9	20.0 - 24.9	25 +	
N	0.08%	1.57%	1.76%	1.39%	0.66%	0.12%	0.00%	0.00%	0.00%	0.00%	**5.6%**
NNE	0.11%	1.72%	2.18%	1.86%	0.78%	0.09%	0.01%	0.00%	0.00%	0.00%	**6.7%**
NE	0.10%	2.85%	3.05%	2.20%	0.84%	0.11%	0.01%	0.00%	0.00%	0.00%	**9.2%**
ENE	0.10%	4.12%	4.66%	2.74%	0.59%	0.05%	0.01%	0.01%	0.00%	0.00%	**12.3%**
E	0.12%	6.10%	6.87%	2.35%	0.26%	0.03%	0.01%	0.01%	0.00%	0.00%	**15.7%**
ESE	0.12%	5.93%	5.16%	1.42%	0.15%	0.04%	0.01%	0.00%	0.00%	0.00%	**12.8%**
SE	0.14%	4.86%	3.02%	0.81%	0.12%	0.03%	0.01%	0.00%	0.00%	0.00%	**9.0%**
SSE	0.11%	3.05%	1.69%	0.48%	0.07%	0.01%	0.01%	0.00%	0.00%	0.00%	**5.4%**
S	0.08%	2.27%	1.19%	0.34%	0.09%	0.02%	0.00%	0.00%	0.00%	0.00%	**4.0%**
SSW	0.07%	1.70%	0.84%	0.21%	0.05%	0.01%	0.00%	0.00%	0.00%	0.00%	**2.9%**
SW	0.07%	1.42%	0.65%	0.18%	0.04%	0.02%	0.00%	0.00%	0.00%	0.00%	**2.4%**
WSW	0.07%	1.28%	0.62%	0.20%	0.05%	0.02%	0.00%	0.00%	0.00%	0.00%	**2.2%**
W	0.07%	1.20%	0.66%	0.24%	0.06%	0.04%	0.00%	0.00%	0.00%	0.00%	**2.3%**
WNW	0.05%	0.99%	0.69%	0.34%	0.14%	0.05%	0.01%	0.00%	0.00%	0.00%	**2.3%**
NW	0.05%	1.22%	0.93%	0.61%	0.35%	0.09%	0.01%	0.00%	0.00%	0.00%	**3.3%**
NNW	0.06%	1.33%	1.12%	0.88%	0.45%	0.10%	0.00%	0.00%	0.00%	0.00%	**3.9%**
TOTAL	**1.4%**	**41.6%**	**35.1%**	**16.3%**	**4.7%**	**0.8%**	**0.1%**	**0.0%**	**0.0%**	**0.0%**	**100%**

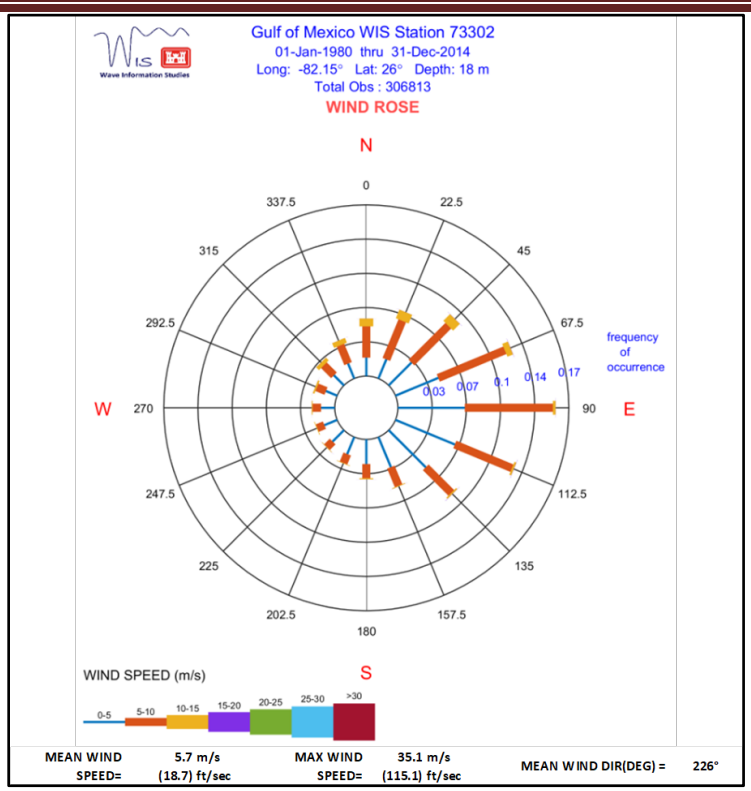


Figure 3.7. Image of Deep Water Wind Rose (from WIS Data).

Table 3.6. Deep water percent occurrence of wind speed and direction (from WIS data)

WIND DIR	PERCENT OCCURRENCE OF WIND SPEED AND DIRECTION (M/S)										TOTAL
	<2.5	2.5 - 4.9	5.0 - 7.4	7.5 - 9.9	10.0 - 12.4	12.5 - 14.9	15.0 - 17.4	17.5 - 19.9	20.0 - 24.9	25 +	
N	0.07%	1.69%	1.71%	1.39%	0.67%	0.14%	0.01%	0.00%	0.00%	0.00%	**5.7%**
NNE	0.07%	1.90%	2.18%	1.95%	0.87%	0.11%	0.01%	0.00%	0.00%	0.00%	**7.1%**
NE	0.06%	2.99%	2.97%	2.23%	0.89%	0.11%	0.01%	0.00%	0.00%	0.00%	**9.3%**
ENE	0.07%	4.32%	4.56%	2.69%	0.58%	0.06%	0.01%	0.01%	0.00%	0.00%	**12.3%**
E	0.07%	6.27%	6.60%	2.33%	0.27%	0.03%	0.01%	0.01%	0.00%	0.00%	**15.6%**
ESE	0.06%	6.04%	4.88%	1.44%	0.17%	0.04%	0.01%	0.01%	0.00%	0.00%	**12.7%**
SE	0.07%	4.95%	2.88%	0.82%	0.14%	0.03%	0.01%	0.00%	0.00%	0.00%	**8.9%**
SSE	0.06%	3.06%	1.60%	0.46%	0.07%	0.01%	0.01%	0.00%	0.00%	0.00%	**5.3%**
S	0.07%	2.29%	1.13%	0.32%	0.10%	0.02%	0.01%	0.00%	0.00%	0.00%	**3.9%**
SSW	0.07%	1.70%	0.79%	0.20%	0.05%	0.01%	0.00%	0.00%	0.00%	0.00%	**2.8%**
SW	0.08%	1.47%	0.63%	0.18%	0.04%	0.01%	0.01%	0.00%	0.00%	0.00%	**2.4%**
WSW	0.05%	1.28%	0.59%	0.19%	0.05%	0.02%	0.00%	0.00%	0.00%	0.00%	**2.2%**
W	0.06%	1.26%	0.64%	0.22%	0.06%	0.03%	0.00%	0.00%	0.00%	0.00%	**2.3%**
WNW	0.06%	1.10%	0.67%	0.32%	0.13%	0.05%	0.01%	0.00%	0.00%	0.00%	**2.3%**
NW	0.06%	1.30%	0.87%	0.56%	0.32%	0.09%	0.01%	0.00%	0.00%	0.00%	**3.2%**
NNW	0.05%	1.48%	1.12%	0.87%	0.43%	0.10%	0.01%	0.00%	0.00%	0.00%	**4.1%**
TOTAL	**1.0%**	**43.1%**	**33.8%**	**16.2%**	**4.8%**	**0.9%**	**0.1%**	**0.0%**	**0.0%**	**0.0%**	**100%**

3.4. ASTRONOMICAL TIDES & WATER LEVELS

3.4.1. ASTRONOMICAL TIDES

The most extensive and continuous data record available for the Collier County is the National Oceanographic and Atmospheric Administration (NOAA) Naples, FL tide Gauge (Station ID: 8725110) located on the Naples City Pier at the West end of 12 Avenue South. (Figure 3.8 & Figure 3.9).

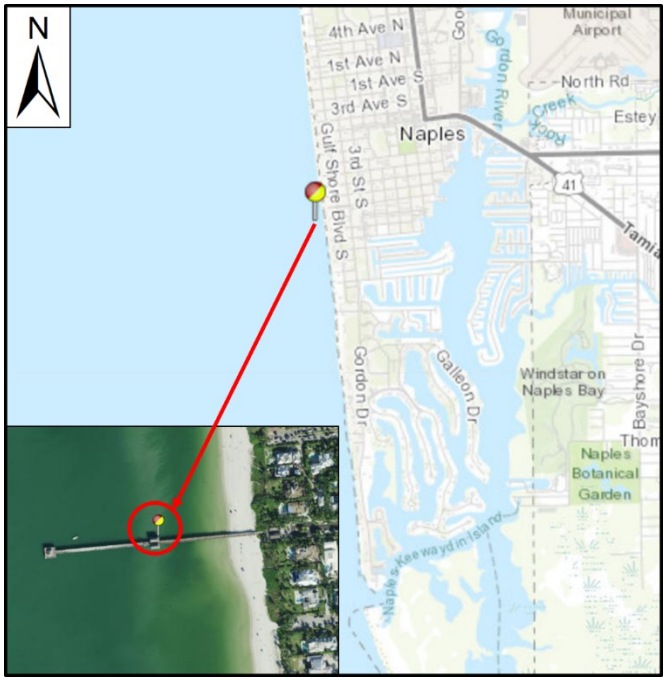


Figure 3.8. Image of NOAA Naples City Pier Gauge location in Naples, FL

Figure 3.9. Picture of NOAA Naples City Point Gauge

Historical trends are determined using measurement data from tide gauge records. All tide levels of the study area have increased from the year that the tide level was initially recorded and account for past, present and anticipated future increases in sea level. The tides in the study area are semi-diurnal, with a mean tide range of approximately 2.01 feet at the Naples tide Gauge (Figure 3.10) and though the tide range during spring tides can exceed 4 feet.

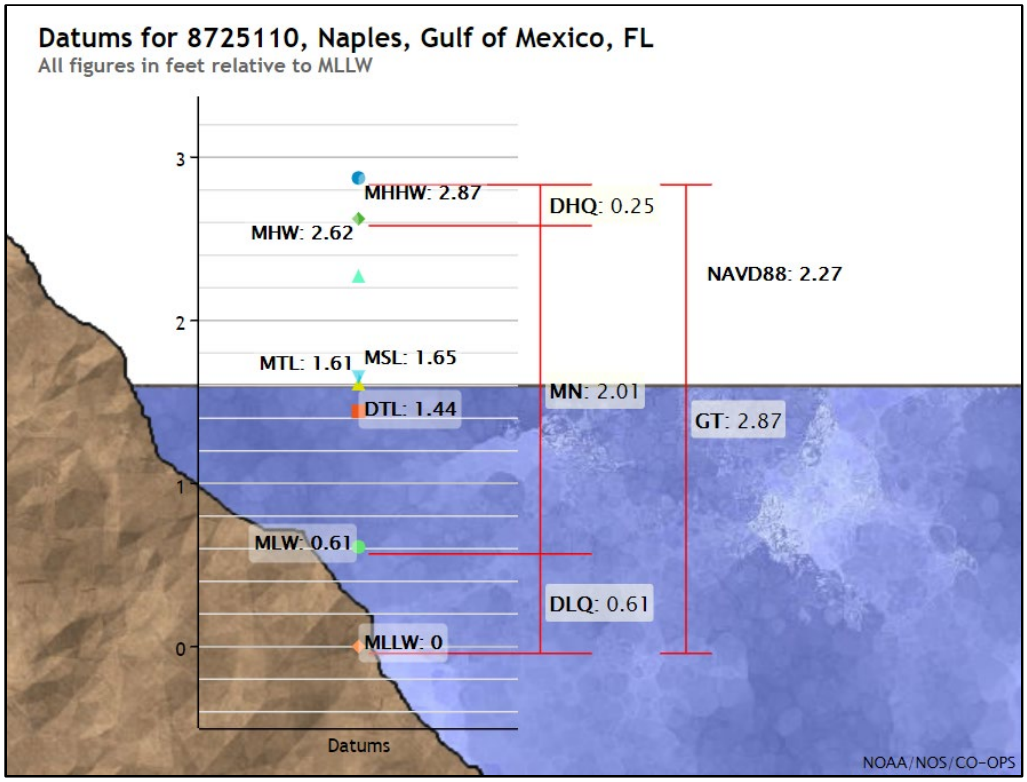


Figure 3.10. Image of the Tide Range.

3.4.2. WATER LEVELS

The tide gauge extreme water level records go back a little over 50 years. Figure 3.11 shows the NOAA Naples tide gauge monthly highest and lowest water levels with the 1%, 10%, 50%, and 99% annual exceedance probability levels in red, orange, green, and blue. The plotted values are in meters relative to the Mean Higher High Water (MHHW) or Mean Lower Low Water (MLLW) datum's established by NOAA (1 foot ≈ 0.3 meters). On average, the 1% level (red) will be exceeded in one year per century, the 10% level (orange) will be exceeded in ten years per century, and the 50% level (green) will be exceeded in fifty years per century. The 99% level (blue) will be exceeded in all but one year per century.

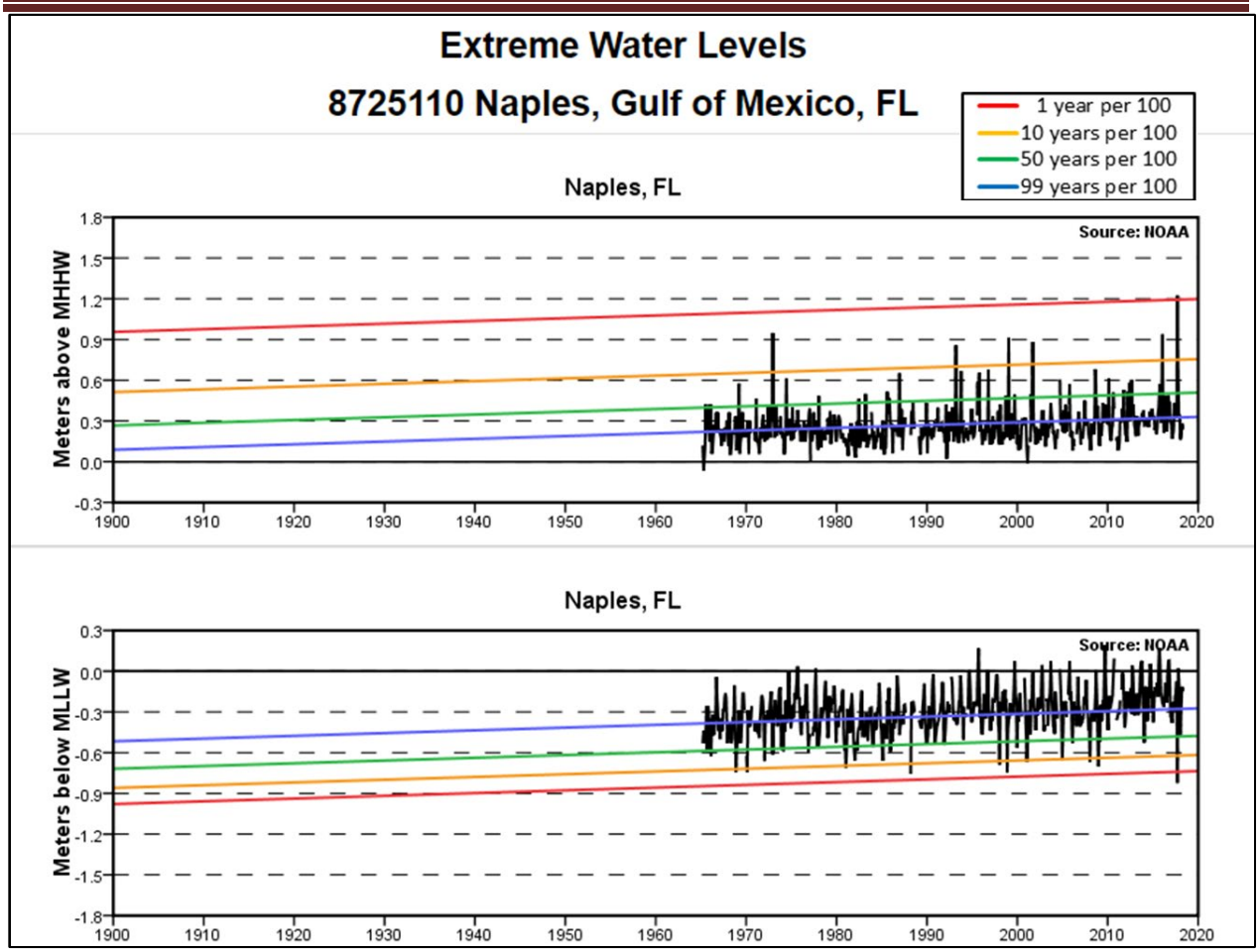


Figure 3.11. Extreme Water Levels measured in Naples City Pier Gauge.

The monthly extreme water levels include the sea level trend of 0.00935 feet per year (2.85 millimeters/year) with a 95% confidence interval of +/- 0.6 millimeters/year. The trend is based on the monthly MSL data from 1965 to 2006 which is equivalent to a change of 0.66 feet in 100 years.

3.5. STORMS

Two general types of major storms affect Collier County in the form of tropical storms and hurricanes. Superstorms also have the potential to impact the area as seen with the "Storm of the Century" in 1993 (see Table 3.8).

3.5.1. TROPICAL STORMS

The term "tropical storm" is applied to a localized intense cyclonic circulation, intense low pressure wind system, forming over tropical oceans and with high winds above 39 miles per hour (mph) to 73 mph, heavy rainfall, large waves, and tidal surges.

3.5.2. HURRICANES

The term "hurricane" is applied to an intense cyclonic storm originating in the tropical and subtropical latitudes of the Atlantic Ocean north of the equator. These storms normally gain intensity as they move over water in the southern latitudes and decay or decrease in intensity as they pass over land or move into the northern latitudes, where conditions are such that the energy of the storm cannot be maintained. A hurricane is characterized by low barometric pressure high winds (over 74 mph), heavy rainfall, large waves, and tidal surges.

Intensity of hurricanes are classified and rated based on the Saffir-Simpson Hurricane Scale. A 1-5 rating system is used where Category 5 being the most intense and Category 1 being the least intense (Table 3.7).

Table 3.7 Saffir-Simpson Hurricane Scale

CATEGORY	WIND (MPH)	DAMAGE POTENTIAL
5	> 156	Catastrophic
4	131 – 155	Extreme
3	111 – 130	Extensive
2	96 - 110	Moderate
1	74 - 95	Minimal

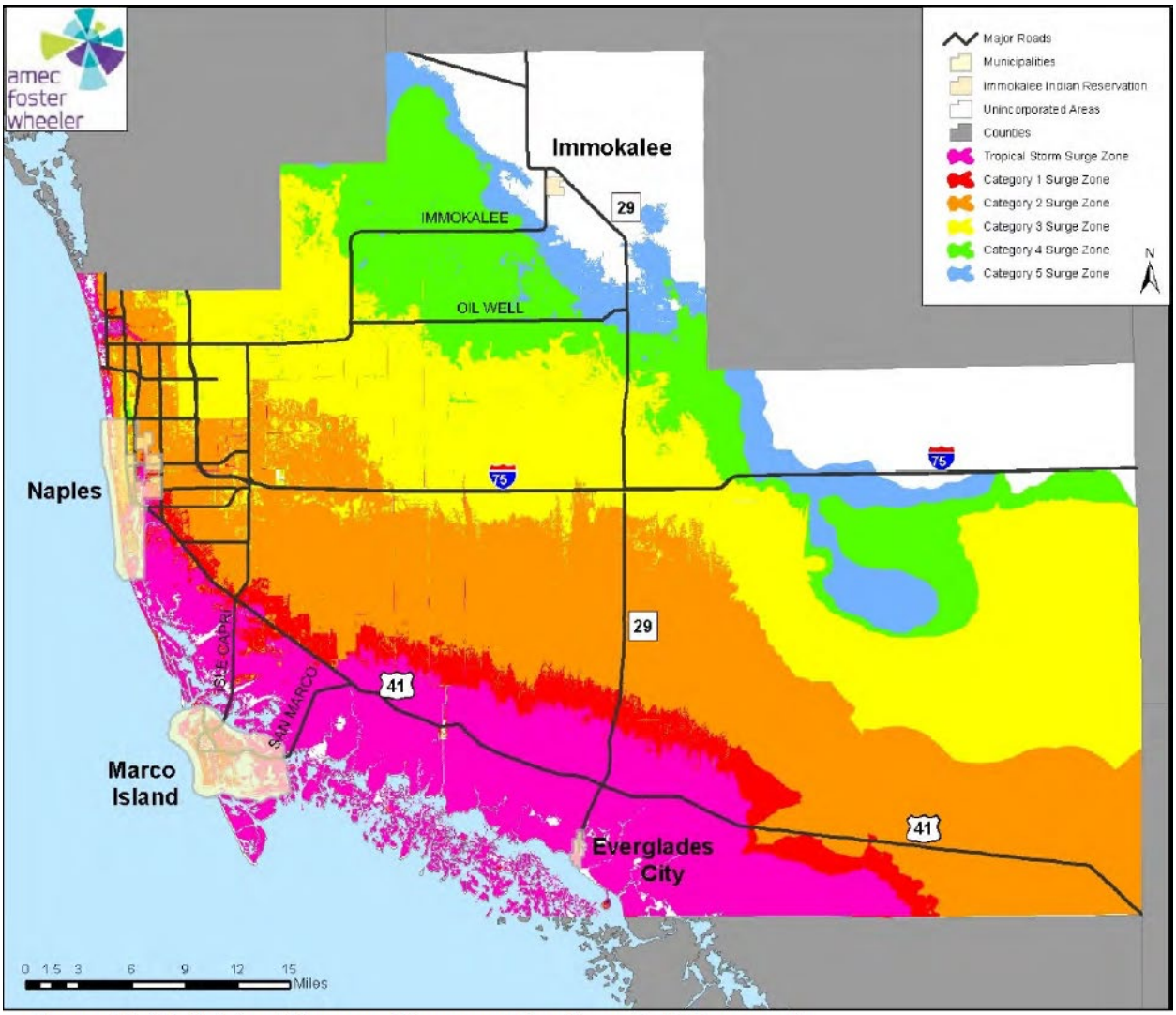
3.5.1. SUPERSTORMS

Superstorm is a term used to classify very large, powerful, and destructive storm systems without another meteorological classification. These storms can produce flooding and high winds of similar magnitude to tropical storms and hurricanes. An example is the 1993 low pressure system often referred to as "The Storm of the Century" which resulted in record snowstorm over the north eastern US and coastal flooding along the Gulf of Mexico coastlines.

3.5.2. STORM SURGE VULNERABILITY

Anticipated SLOSH model surge elevations for a tropical storm and category 1-5 hurricanes are shown in Figure 3.12. The Sea, Lake and Overland Surges from Hurricanes (SLOSH) model is a computerized numerical model developed by the National Weather Service (NWS) to estimate storm surge heights resulting from historical, hypothetical, or predicted hurricanes by taking into account the atmospheric pressure, size, forward speed, and track data. The feature set depicting surge zones in this figure was created using a Surge Modeling application created for the Florida Statewide Regional Evacuation Update Study. The data was derived from National Hurricane Center SLOSH model runs on all the NOAA SLOSH basins throughout Florida. The runs create outputs for all different storm simulations from all points of the compass. Each

direction has a MEOW (maximum envelope of water) for each category of storm (1-5), and all directions combined result in a MOMs (maximum of maximums) set of data. The MOMs are used in Figure 3.12 (Collier County 2015). The figure shows how susceptible the County is storm surge flooding, even from "lower" level storms such as Tropical Storms and Category 1 hurricanes.



Source: Florida Division of Emergency Management (http://www.floridadisaster.org)

Figure 3.12 Storm Surge Zones for Collier County (Collier County 2015)

3.5.3. HISTORICAL STORMS

Table 3.8 displays the date of historical storm events, the type of storm, and the peak water surfaces elevations if available. For storms occurring since 1965, the peak water surface elevations shown were measured by the NOAA – Naples Pier tide gauge while older storm water surface elevation were gathered from previous reports and studies if available. All elevations are referenced to NAVD 88. Hurricane Donna, the

storm in Table 3.8 with the highest water level, ranks as one of the great storms of the 20th century. Its center traveled north, paralleling the Gulf Coast west of Collier County. As the center moved northward, southwesterly winds generated high tides that flooded most of Goodland, Marco, and Naples (FEMA 2012). Hurricane Donna's track is shown in Figure 3.13.

Table 3.8 Historical Storm Events

	Storm Event – Date & Name	**Type of Storm**	**Peak Water Surface Elevations (in feet)**
1	October 7, 1873	Hurricane	7.5 – 10.5
2	September 25, 1894	Hurricane	9.5 – 10.5
3	September 1926 (Great Miami Hurricane)	Hurricane	4.7
4	August/September 1935 (Great Labor Day Hurricane)	Hurricane	N/A
5	October 1944 (No Name)	Hurricane	6
6	August/September 1960 (Donna)	Hurricane	10.3
7	October 1964 (Isbell)	Hurricane	N/A
8	August/September 1965 (Betsy)	Hurricane	1.93
9	August 1981 (Dennis)	Hurricane	1.54
10	July 1985 (Bob)	Hurricane	1.78
11	August 1992 (Andrew)	Hurricane	1.24
12	March 1993 ("Storm of the Century")	Superstorm	3.33
13	November 1994 (Gordon)	Hurricane	1.43
14	October/November 1998 (Mitch)	Hurricane	1.48
15	September 1992 (Harvey)	Tropical Storm	2.22
16	August 2004 (Charley)	Hurricane	No gauge data available
17	October 2005 (Wilma)	Hurricane	2.47
18	August 2008 (Fay)	Tropical Storm	1.22
19	June 2012 (Debby)	Tropical Storm	2.18
20	August 2012 (Isaac)	Tropical Storm	2.62
21	August/September 2017 (Irma)	Hurricane	4.6

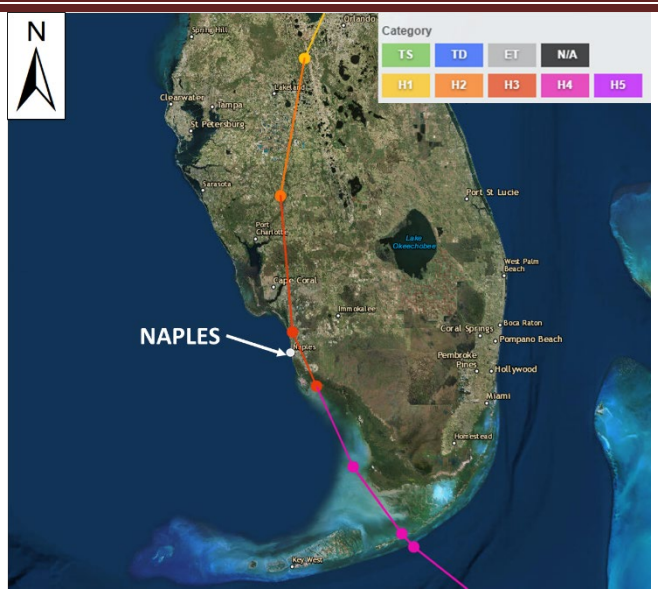


Figure 3.13. 1960 Hurricane Donna track

Figure 3.14 thru Figure 3.19 are plots of the water surface elevation (the predicted versus the verified water levels) and wind speeds (if available) measured at the NOAA – Naples Pier Gauge, during some of the storm events shown in Table 3.8. The peaks shown in the figures are what is shown in the Table 3.8.

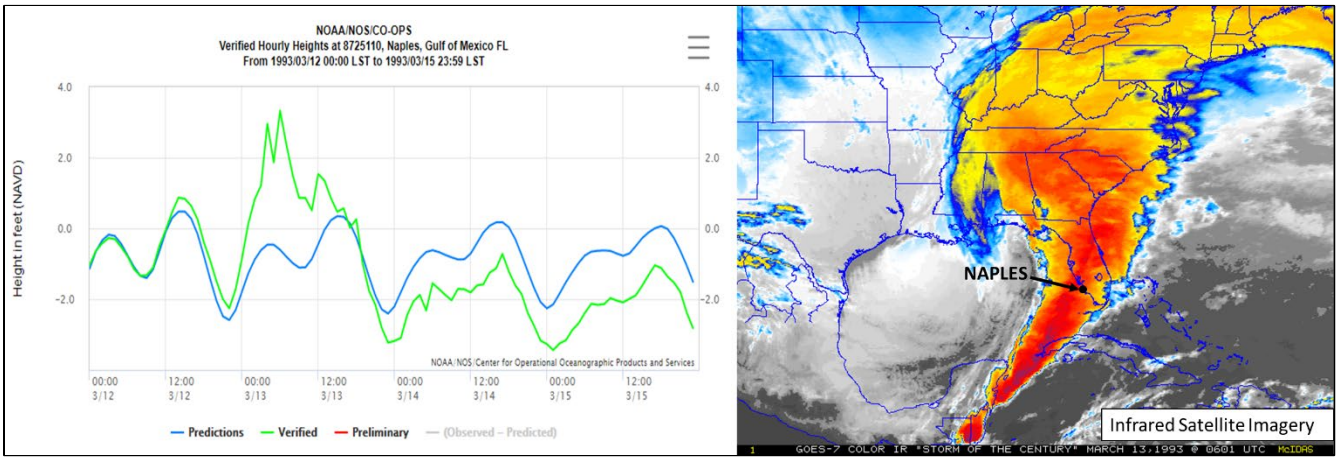


Figure 3.14. Predicted vs. Verified water levels during "Storm of the Century" in 1993 (Figure Source: **https://www.ncdc.noaa.gov/news/remembering-storm-century-20-years-later**)

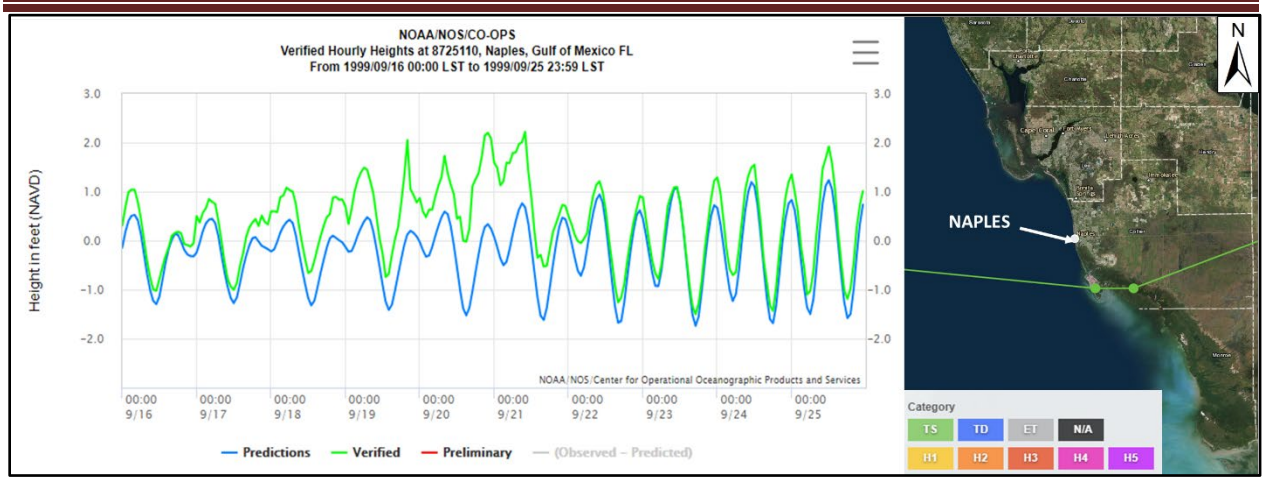


Figure 3.15. Predicted vs. Verified water levels during Tropical Storm Harvey in 1999.

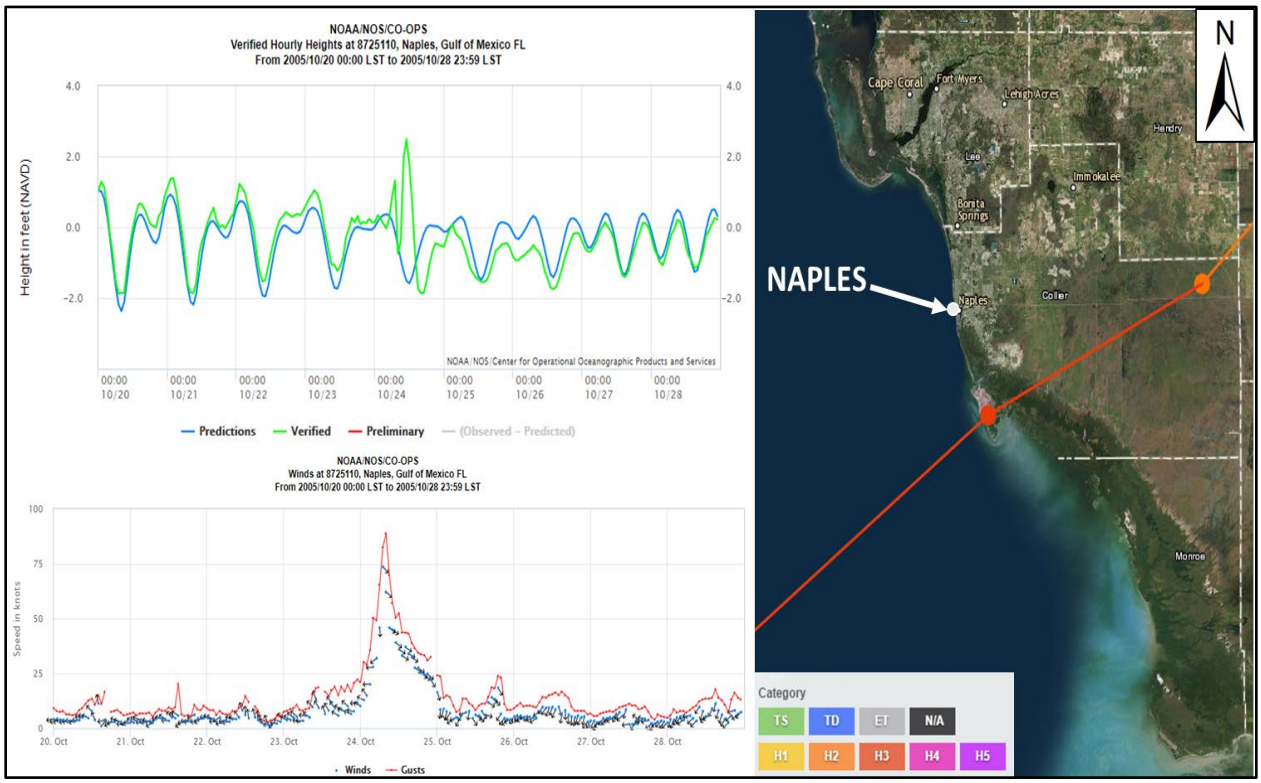


Figure 3.16. Predicted vs. Verified water levels and recorded wind speeds during Hurricane Wilma in 2004.

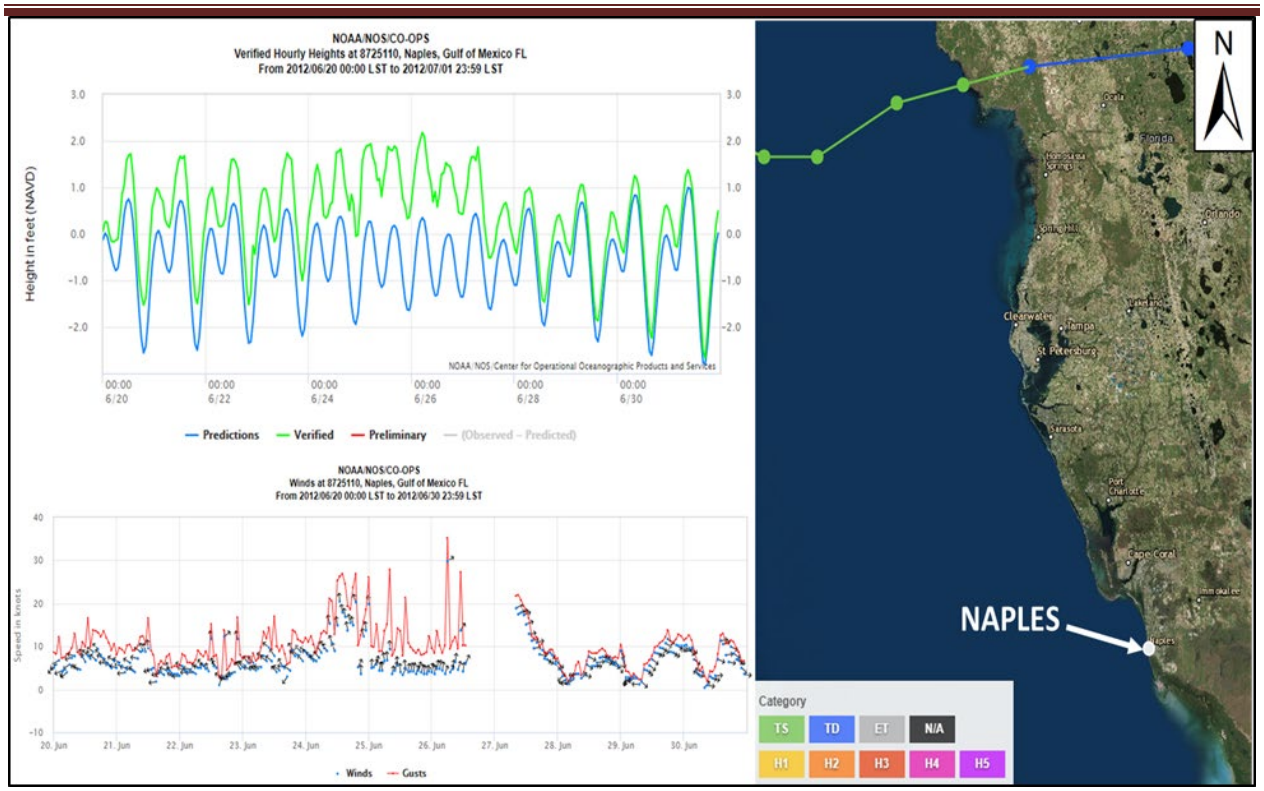


Figure 3.17. Predicted vs. Verified water levels and recorded wind speeds during Tropical Storm Debby in 2012**.**

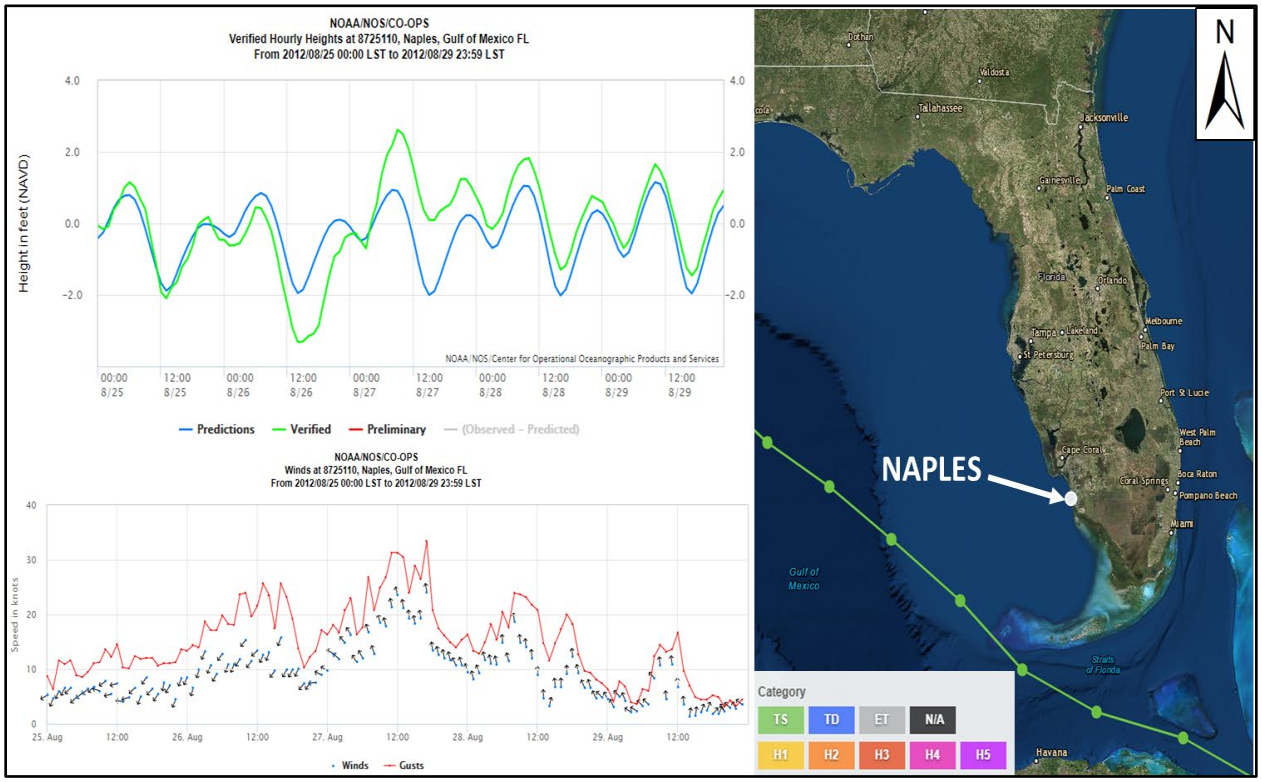


Figure 3.18 Predicted vs. Verified water levels and recorded wind speeds during Tropical Storm

Isaac in 2012.

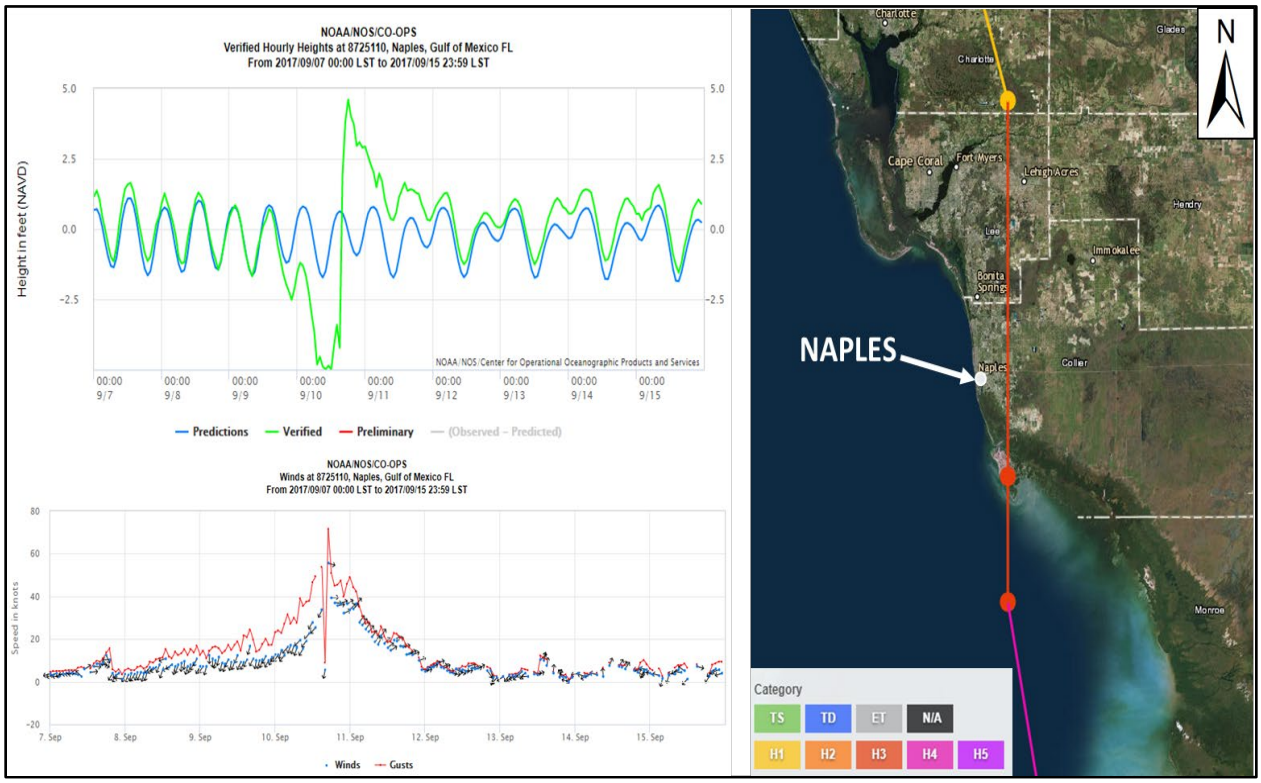


Figure 3.19. Predicted vs. Verified water levels and recorded wind speeds during Hurricane Irma in 2017.

3.6. SEA LEVEL RISE & LAND SUBSIDENCE

Sea Level Rise is an increase in the volume of water in the world's ocean, resulting in an increase in sea level called global sea level rise. The sea level rise local to a specific area is called relative sea level rise. Sea level rise at specific locations (relative sea level rise) may be more or less than the global average (global sea level rise). Sea level rise is attributed to global climate change by the added water from melting ice sheets and glaciers. Melting of floating ice shelves or icebergs at sea raises sea levels only slightly. Local factors such as subsidence of the land also impact local communities. Subsidence is the motion of the land surface as it shifts downward relative to a vertical datum.

3.6.1. RELATIVE SEA LEVEL RISE (RSLR)

RSLR is predicted to continue in the future as the global climate changes. According to National Oceanographic and Atmospheric Administration (NOAA), RSLR for Collier County averages 0.00935 feet/year (2.8 mm/year) for the Naples tide Gauge (Figure 3.20). This equates to approximately one-half foot to three-quarters foot of Relative Sea Level Rise (RSLR) over the next 70 years. Additionally, USACE recently

issued Engineer Regulation 1100-2-8162, "Incorporating Sea Level Change in Civil Works Programs." This guidance document provides three different accelerating eustatic, (worldwide changes in sea level) SLR scenarios: a conservative scenario (historic rate of sea level change), an intermediate scenario and a high scenario. The scenarios presented in the USACE Sea Level Change Curve Calculator, estimate RSLR from the year 2028 thru 2077 (assuming a 50 year project life), for Collier County at the NOAA's Naples. FL Gauge, to be 0.14 feet for the low approach, 0.78 feet for the intermediate approach and 2.82 feet for the high scenario. The project Sea Level Rise Curves are presented in Figure 3.21.

Figure 3.20 shows a plot of the monthly mean sea level, at the NOAA – Naples, FL Gauge from 1900 – 2020, with the average seasonal cycle removed and monthly average (represented by the blue curve), and the linear trend with its 95% confidence interval which was obtained after accounting for the average seasonal cycle. The plotted values are relative to 1983-2001 mean sea level datum.

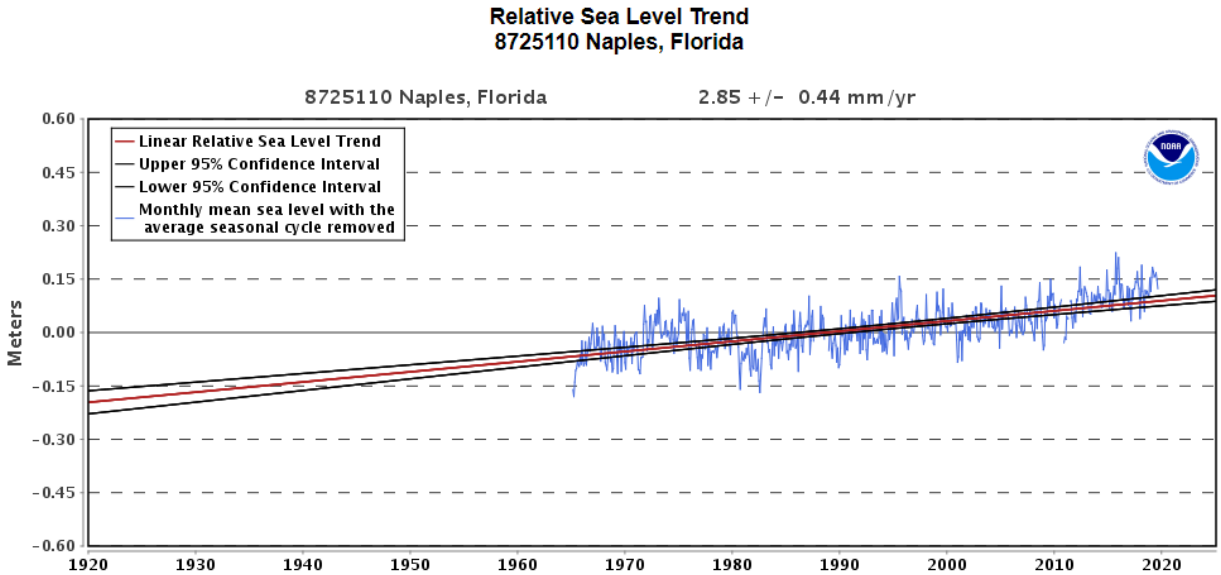


Figure 3.20. NOAA Sea Level Trend at Naples, FL (accessed December 2019)

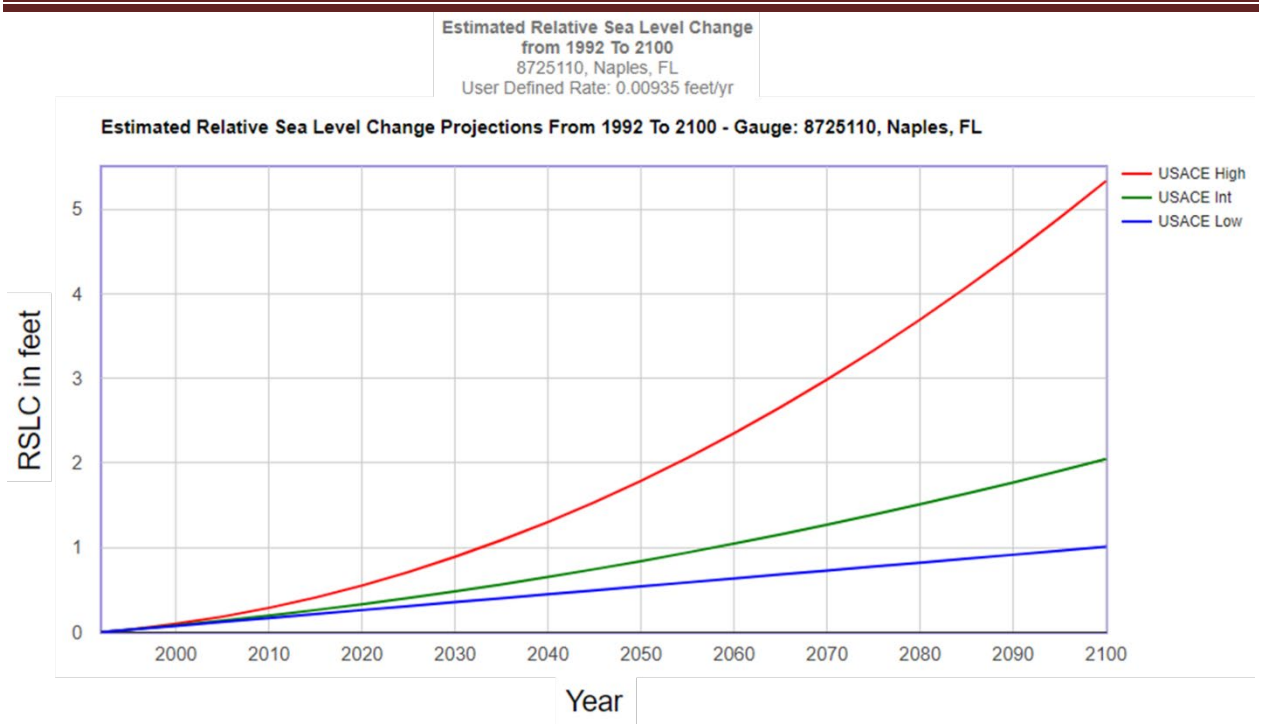


Figure 3.21. USACE Project Sea Level Curves

More detail on the tides and SLR and their influence on water levels in Collier County will be discussed in the Hydraulics, Hydrology & Coastal Engineering Sub-Appendix.

3.6.2. LAND SUBSIDENCE

No existing literature was located on land subsidence in Collier County. It was stated by County Officials that Collier County does not experience land subsidence.

3.7. SEDIMENT TRANSPORT

3.7.1. NORTH COUNTY BEACHES

A sediment budget estimates the sand movement during a specific time frame for a period over a particular segment of coast. Collier County developed a sediment budget (Figure 3.22) after its most recent dredging renourishment based on post construction monitoring results from 2006 to 2009 surveys and sediment bypassing information from inlets.

The sediment transport is generally towards the south. At the inlets, sediment naturally moves into the inlet, but is bypasses through dredging and natural processes. Overall, 300 CY per year is shown to enter the system at Barefoot Beach near R-10 and approximately 900 CY per year leaves the system near Gordon Pass (CPE, 2011).

From this sediment budget is it seen that Naples Beach is relatively stable, except just south of Doctors Pass inlet, while Vanderbilt and Park Shore have an erosion trend. The

dredge and fill arrows show that the sediment bypassing has been beneficial, supporting accreting areas wherever sand is bypassed, except south of Clam Pass (CPE, 2011).

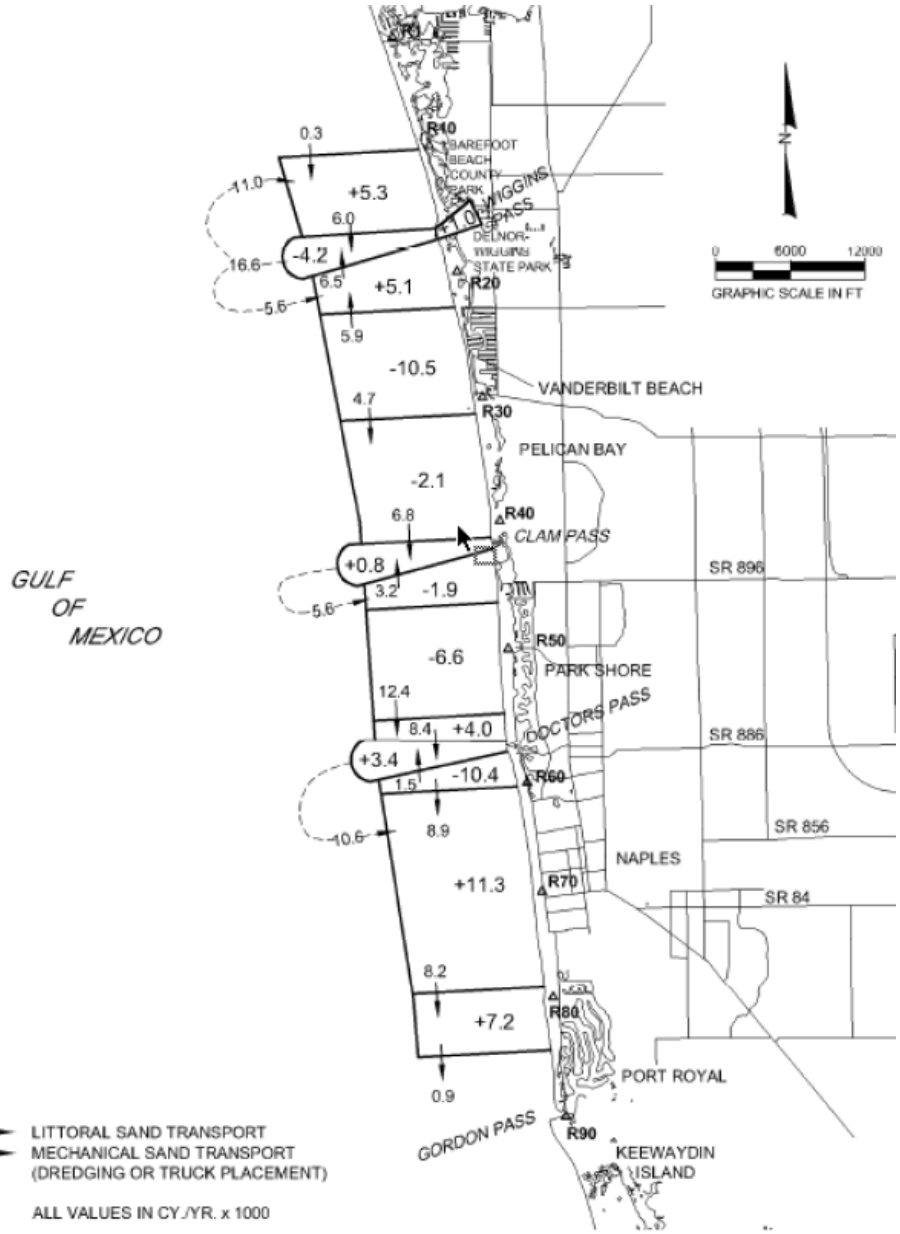


Figure 3.22 Sediment Budget for Collier County, FL (2006-2009) (CPE, 2011)

There are several "hot spot" areas, or areas with higher erosion rates along the North County Beaches. Some of these areas have existed since the initial 1996 beach nourishment while others have evolved over times. Hot spots are primarily a result of inlet effects, gaps in the hardbottom, and existing groin influences. These areas include: North Wiggins Pass (Barefoot Beach) from monument R-14 to R-16, South of Wiggins Pass from R-18 to R-20, Vanderbilt Beach in the vicinity of R-27, Pelican Bay between

R-34 and R-36, South Clam Pass between R-42 to R-44, North Park Shore between R-45 and R-48, Park Shore between R-51 and R-53, South Doctors Pass from R-58A to R-59, south of Lowdermilk Park near R-62 and R-63, and South Naples Beach between R-70 and R-72 (CPE 2011). This information helped to further organize beach reaches by separating areas with different erosion rates.

3.7.2. MARCO ISLAND

Marco Island is a low, wave-built, sandy barrier island aligned with the regional coastline. Its shoreline varies from a northwesterly orientation along its northern segment to a north-south orientation along its southern segment. Marco Island lies between the Big Marco – Capri Pass dual inlet system in the north and Caxambas Pass to the South. South of Caxambas Pass is Kice Island, an uninhabited barrier island. Further to the south and east lie the Ten Thousand Islands and Cape Romano, the end of the Florida Gulf Coast barrier island chain. Today the beachfront is almost completely developed with multi-story condominiums and hotels.

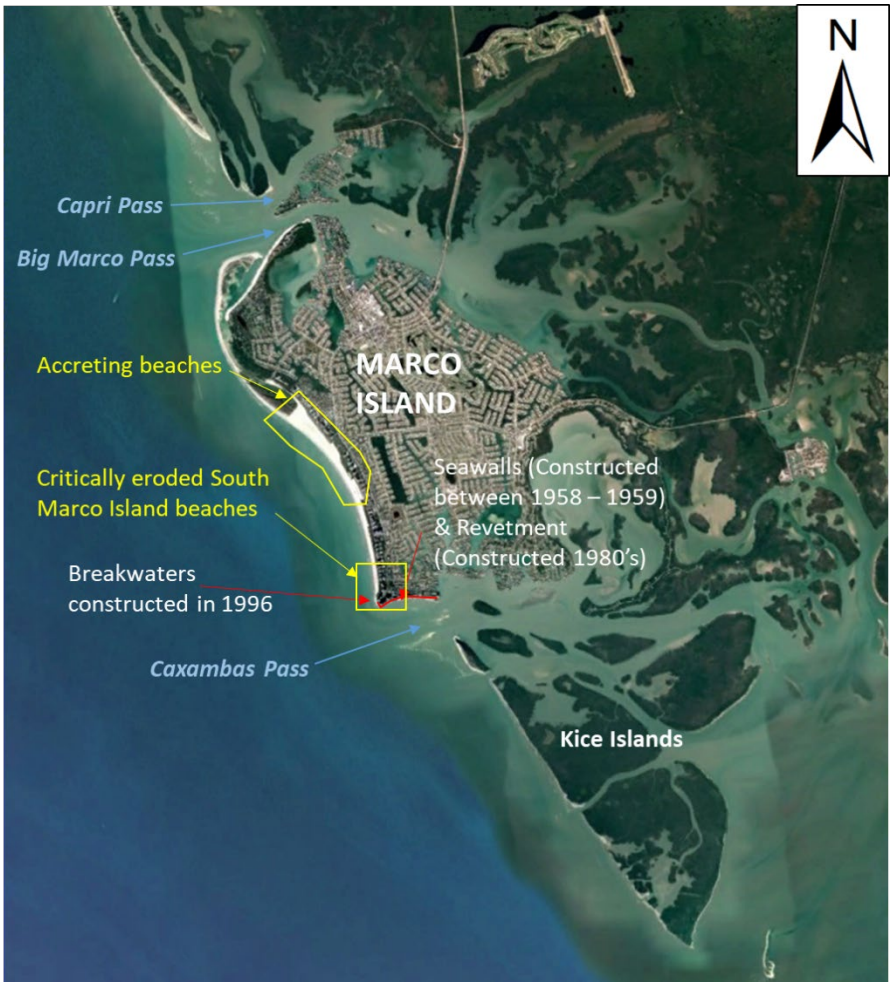


Figure 3.23 Marco Island and surrounding features

Prior to 1960, Caxambas Pass ebb delta shoals, which stored southerly directed sediment in the wide, shallow, offshore platform, acted as a feeder berm that nourished the central and southern Marco Island beaches during periods of northerly directed transport. Anthropogenic activities disrupted this natural northern transport when seawalls were constructed in 1958-1959 along the southern end of Marco Island. Sand which would have been transported north was deflected off the seawalls and transported south, overnourishing the updrift side of Caxambas Pass ebb delta, and resulting in the formation of an approximate 2,500 ft long intertidal spit. The spit redirected flow from Caxambas Pass and caused the ebb delta to empty due south and westerly tidal currents were completely blocked by sediment bypassing the seawall. As a result, sediment eroded rapidly from the South Marco Island beaches and deposited on the Caxambas Pass tidal delta shoals. Over the past decades groins, detached breakwaters, and beach nourishment have reduced the high erosion rates on the southern end, but this is still a critically eroded beach area (CEC, 2011).

The northern Marco Island beaches accrete as a result of southerly sediment transport from Big Marco Pass and, when sediment transport is northerly, sediment from the southern beaches. A sediment budget for Marco Island is present in Figure 3.24.

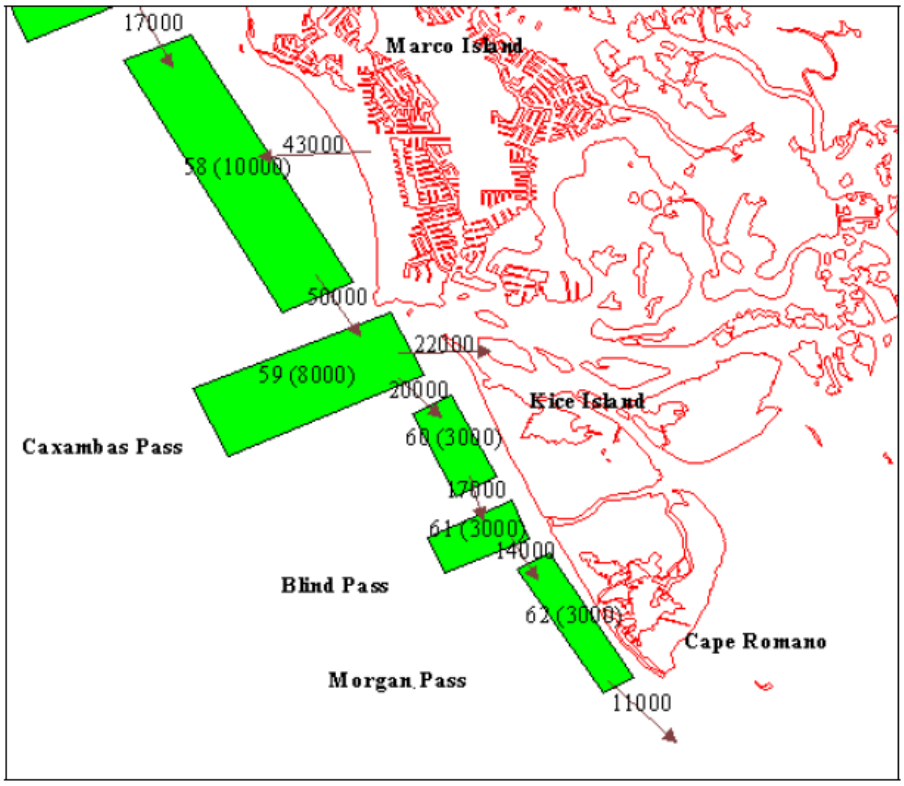


Figure 3.24 Southern Collier County Budget (Taylor Engineering 2002)

3.8. SHORELINE ARMOR AND STRUCTURE IMPACTS

Throughout Collier County there are several existing rock structures and armored areas. These structures are portrayed in Figure 3.25 to Figure 3.32. The impacts and status of these structures will be discussed in the following paragraphs.

Figure 3.25 Existing groins located near Seagate Drive and Park Shore Beach (image from Google Earth)

Figure 3.26 Existing Doctor's Pass Jetties and Breakwaters (image from Google Earth)

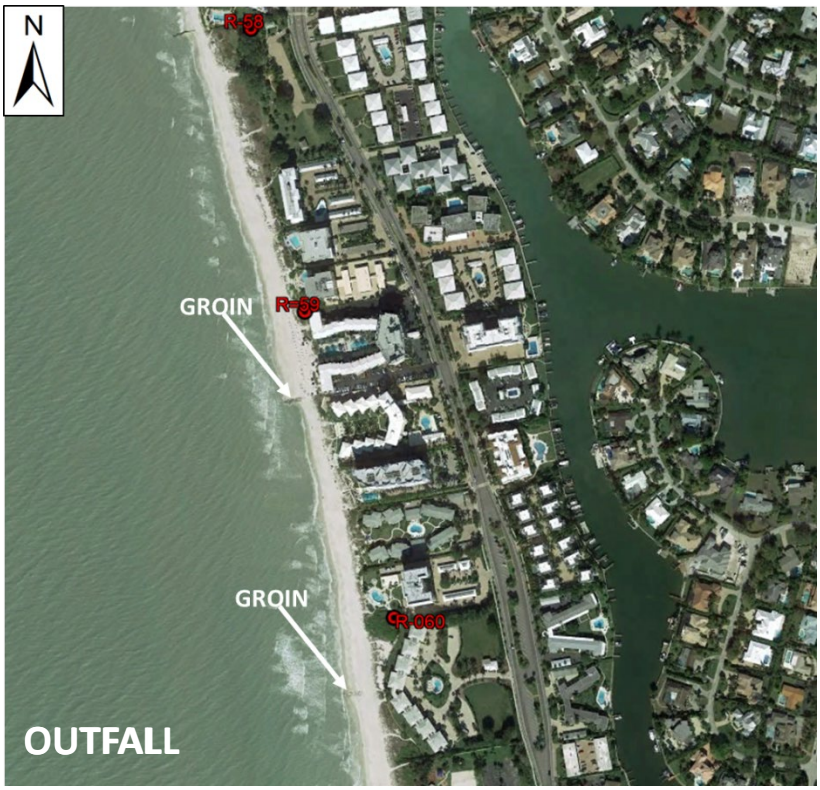


Figure 3.27 Existing groins along Naples Beach (image from Google Earth)

Figure 3.28 Existing groins/outfalls along Naples Beach (image from Google Earth)

Figure 3.29 Existing groins/outfalls along Naples Beach to be removed prior to project construction. See Paragraph 3.8.1 GROINS/OUTFALLS (image from Google Earth)

Figure 3.30 Existing groins/outfalls along Naples Beach to be removed prior to project construction. See Paragraph 3.8.1 GROINS/OUTFALLS (image from Google Earth)

Figure 3.31 Existing structures near Gordon Pass (image from Google Earth)

Figure 3.32 Existing structures in the south of Marco Island (image from Google Earth)

3.8.1. GROINS/OUTFALLS

On Naples Beach, there are many groin/outfall structures along the coast between

Doctors Pass and Gordon Pass. There are no groins on Vanderbilt Beach or Barefoot Beach while Park Shore has three small groin like structures in the vicinity of Seagate Drive (R-45)(CPE, 2011). Generally, groins cause a localized offset fillet in the shoreline that has a very focused area of benefit and impact that is note visible monitoring at 1000 ft increments, (distance between the Florida R-monuments). A numerical modeling study performed in 2011 compared with and without structure conditions at Naples and Park Shore Beaches. The results indicated that the influence of the structures is significant to behavior and beach fill template tend to perform better without the structures. With a robust nourishment and inlet bypassing program, the benefits groins diminish, especially in an area with a weak long shore transport direction and magnitudes (CPE, 2011b). The City of Naples is currently undergoing the permitting for a stormwater improvement project which will remove five (5) beach outfalls along Naples Beach (ECE, 2018).

3.8.2. DOCTORS PASS STRUCTURES

Doctors Pass is the northernmost stabilized inlet in Collier County located between R-57 and R-58. The stone jetties were constructed in 1960 for stabilization when the pass was widened. The area downdrift of Doctors Pass has experienced a high rate of erosion, due changes in inlet management and ebb shoal shrinkage. The hotspot extends from the jetty to R-58. In 2018, several structures were constructed at Doctors Pass, including a spur extension on the southern jetty, an offshore breakwater, and a detached groin (Figure 3.26) after a numerical modeling study suggested the additional rock structures to reduce erosion (CPE, 2011b). The 2018 inlet maintenance dredging project also included a change in management practices which extended the placement of sand dredged in the inlet to include the hotspot area (CPE, 2011).

3.8.3. GORDON PASS STRUCTURES

Gordon Pass is another stabilized inlet in Collier County. The pass was originally widened and the south jetty was constructed for stabilization in 1962. The north jetty was constructed in 1977 and lengthened in 1987. The navigation channel from Gordon Pass to Naples is the only Federal Navigation Channel within Collier County with disposal of the dredged maintenance material to the south of the inlet on Keewaydin Island (CPE, 2011). The rock structures north Gordon Pass (Figure 3.31) were constructed the 1950's and 1960's, followed by an installation of seawalls to protect against high levels of erosion. During the 1940's and 1950's upland and canal developments along the interior bay system increased the bay area and tidal prism flowing though Gordon. It is theorized that an increase in the Gordon Pass ebb shoal capacity towards a larger equilibrium than its size prior to develop may have been a factor in increased beach erosion north of Gordon Pass which resulted in the construction of the various structures (Dabees, 2011).

3.8.4. MARCO ISLAND STRUCTURES

The seawall at the southern end of Marco Island (Figure 3.32) was constructed by the

U.S. Army Corp of Engineers for the Navy between 1958 and 1959, replacing the natural sandy flow boundary with a concrete structure. The construction of the seawall had an immediate impact on the shoreline, with sand deflected and transported south, overnourishing the updrift side of the Caxambas Pass ebb delta, and the southern beachfront rapidly eroding. In order to address the anthropogenic influences of the seawall, several steps were taken by the County (CEC, 2011), The first step was to construct a rock revetment adjacent to the seawall to reduce wave energy in the 1980s. The next step involved restoring South Marco Beach through s major beach nourishment project in 1991. Two short terminal rock groins were constructed perpendicular to the wall to reduce losses and reorient the shoreline. The third step in 1996 involved constructing three detached breakwaters approximately 600 ft seaward of the seawall. The breakwaters were designed to create a salient along the shoreline by managing wave energy and reducing sand losses into Caxambas Pass (CEC, 2011). Monitoring after the breakwater construction showed reduced sand losses along the south nourishment segment by approximately 75% (H&M, 2011b).

3.9. NEARSHORE HARDBOTTOM

Nearshore hardbottom is present along the Collier County Shoreline with resources identified through sidescan sonar survey and diver verification on several occasions. In 2003, a sidescan sonar survey documented approximately 500 acres of nearshore hardbottom formations along approximately 1,000 ft of shoreline between R-17 and R-81. Divers have documented developed hardbottom communities consisting of consisting of scleractinian coral species (*Siderastrea* spp., *Solenastrea* spp., *Oculina* spp. and *Phyllangia americana*), octocorals (*Leptogorgia* spp.), macroalgae (primarily *Gracilaria* spp., *Caulerpa* spp. and *Hypnea* spp.) sponges, tunicates and other benthic invertebrates and fish. The County is required to monitor nearshore hardbottom habitat impacts under Florida Department of Environmental Protection (FDEP) and USACE permits for three years post construction. In compliance with the final FDEP-approved monitoring plan, a pre-construction biological monitoring survey was conducted in 2005, and four post-construction surveys were completed to monitor potential impacts from the 2006 Beach Nourishment Project. Also, as compensation for potential effect from the 2006 project to nearshore hardbottom resources, FDEP required that Collier County construct a 1.09-acre artificial reef. This artificial reef was constructed in 2007, and based on two years of monitoring, FDEP determined the artificial reef had provided successful mitigation and that no further monitoring was required. Most recently, hardbottom was surveyed in 2013, 2015, and 2016 in association with the 2013-14 Beach Nourishment Project (CB&I 2016, Collier County 2018). Hardbottom has been located in the nearshore environments listed in Table 3.9.

Table 3.9 Nearshore Hardbottom Coverage Locations

COUNTY BEACH	FDEP RANGE MONUMENT
Vanderbilt	R-21 to R-38

Park Shore	R-43 to R-55
Naples	R-58 to R-65

Historical analysis of the shoreline erosion rates has shown that areas with gaps in the nearshore hardbottom tend to have higher erosion rates than shorelines bordered by continuous hardbottom (CPE 2011b).

3.10. INLET EFFECTS

Collier County has four inlets located throughout the study area, Wiggins Pass, Clam Pass, Doctors Pass, and Gordon Pass. Inlets interrupt the predominantly southern littoral drift and cause high erosion rates of the adjacent beaches.

CHAPTER 4 FEMA SOUTHWEST FLORIDA (SWFL) STUDY

This chapter briefly discussed the Federal Emergency Management Agency (FEMA) Region IV Southwest Florida Storm Surge Study (SWFL Study) and how the study results were used for the Collier County CSRM Study. The FEMA SWFL Study is discussed in more detail in the Hydraulics, Hydrology, and Coastal Sub-Appendix at the end of this Engineering Appendix.

4.1. PURPOSE OF THE FEMA SWFL STUDY

The FEMA SWFL Study includes the coastal counties of Sarasota, Charlotte, Lee, and Collier and the inland counties of DeSoto and Hendry. The purpose of the FEMA SWFL Study is to determine the flood risk for these coastal areas for production of revised Flood Insurance Rate Maps (FIRMs).

The hydrodynamic modeling output data from the FEMA SWFL Study was provided to the Norfolk PDT and the supplied modeling data was processed to produce statistical stillwater level elevations, statistical wave heights and periods, and storm hydrographs. The statistical analyses resulted in stillwater level elevations and wave heights as average recurrence intervals (ARI) for a 50% flood (2 year flood), 20% flood (5 year flood), 10% flood (10 year flood), 5% flood (20 year flood), 2% flood (50 year flood), 1% flood (100 year flood), 0.50% flood (200 year flood), 0.20% flood (500 year flood), 0.10% flood (1,000 year flood), for different confidence limits.

4.2. FEMA SWFL STUDY STORMS, WATER LEVELS, AND WAVES

To develop the FEMA SWFL Study storms, data from historical storms was used to develop a statistical description of the hurricane storm climate of the area in terms of parameters such as central pressure deficit, radius to maximum winds, forward speed of the storm, azimuth of the storm track, etc., allowing for the probabilistic characterization of the occurrence and characteristics of potential hurricanes that may cause significant flooding along the SWFL coast. Extra-Tropical storms were not included in the FEMA SWFL Study because previous FEMA studies in Florida provided evidence that they do not contribute to 10YR, 50YR, 100YR, 200 YR, 500YR, and 1000YR storm surge heights.

As a result from modeling computed for this study, the modeling data was processed and storm surge, wave height, wave period, and waters levels were produced for frequencies at “Save Points” located throughout Collier County. Processing of the data also provided wave and water level hydrographs for 357 storms the FEMA study used for the production runs of its numerical hydrodynamic model (ADCIRC). The processed water level and wave data from the FEMA SWFL study was utilized as the source for the Collier County CSRM engineering analysis and as the for the meteorological driving forces in the Beach-fx and G2CRM modeling.

It should be noted that the native datum of the FEMA SWFL Study results was based on local mean sea level (MSL) tidal epoch 1983-2001. This was the datum that was used to define the regional hydrodynamic model (ADCIRC), from which storm surge results were obtained. The results of the FEMA SWFL Study water levels were referenced to the year 1992, the

midpoint of the current National Tidal Datum Epoch (NTDE) (1983-2001). Therefore, the difference or increase in the relative local level rise (based on the USACE low scenario) from 1992 to 2019 (the start of the project) was added to bring the water levels to present date. These present data water levels were then adjusted accordingly using built in USACE curves in engineering models to account for SLR to project construction and throughout the 50 year period of economic analysis. The values were also converted from, MSL to NAVD 88.

4.3. HISTORICAL TIDE GAUGE ANALYSIS VS. NUMERICAL MODELING

There have been discussions in the past about computing frequency water levels from a historical tide gage analysis versus computer modeling (in this case the FEMA SWFL Study modeling). The historical record at the NOAA Naples Pier gauge, established in 1965, has no record of the largest events which caused significant storm surge flooding to the Greater Naples Area (see Table 1.1 Historical Storm Events. The highest waters levels recorded since the gauge was established range from 3 - 4.6 ft (NAVD88), with the 4.6 ft elevation occurring during Hurricane Irma in September 2017. These recorded historical maximum water levels are approximately equal to a 10% ACE flood to 5% ACE flood (10 to 20-year) tidal events. A statistical gage analysis of the historical record may suggest that what has occurred in the past will occur in the future, this may underestimate the risk. Modeling effects, such as what was done for the FEMA SWFL Study, provide an opportunity to evaluate impacts of stronger hypothetical storms that may not have occurred on record, but could occur.

CHAPTER 5 ENGINEERING EVALUTION

5.1. GENERAL

This chapter briefly discusses the analysis, design, and calculations and considerations that were performed for the structural measures for the Collier County CSRM. More detail can be read in the sub-appendices.

5.2. PROJECT ALIGNMENT

The term "Project Alignment" refers to structural methods (floodwall, surge barrier, etc.) that serve as a barrier to water flow. For the Collier County CSRM, structural method also include beach berms and dunes. Figure 5.1 and Figure 5.6 are images of the proposed project alignment with the all the initial proposed structural measures though out North Collier County and Marco Island respectively along with their associated Planning Areas used for plan formulation and economics analysis. Note that non-structural measures are also being reviewed for this study and are not reflected in this engineering appendix. For Planning Areas 2 and 4, the only structural measure is the beach and dune while the remainder of the Planning Areas not protected by the beach and dune are non-structural only. For Planning Area 1, 3, and 5 which contain barrier and wall structures, both structural and non-structural measures were evaluated in the back bay areas not protected by the beach and dunes. Figure 5.1and Figure 5.6 show all the areas considered, but not all Planning areas are in the Tentatively Selected Plan (TSP). The TSP is the NED plan and Alternative 4 from the final array of alternatives. However since PA4 was not incrementally justified as a separable element, it is not included in the TSP. The TSP is shown Figure 5.7.

The following paragraphs detail the modeling which occurred to screen out planning areas and structural/non-structural measures to develop the TSP.

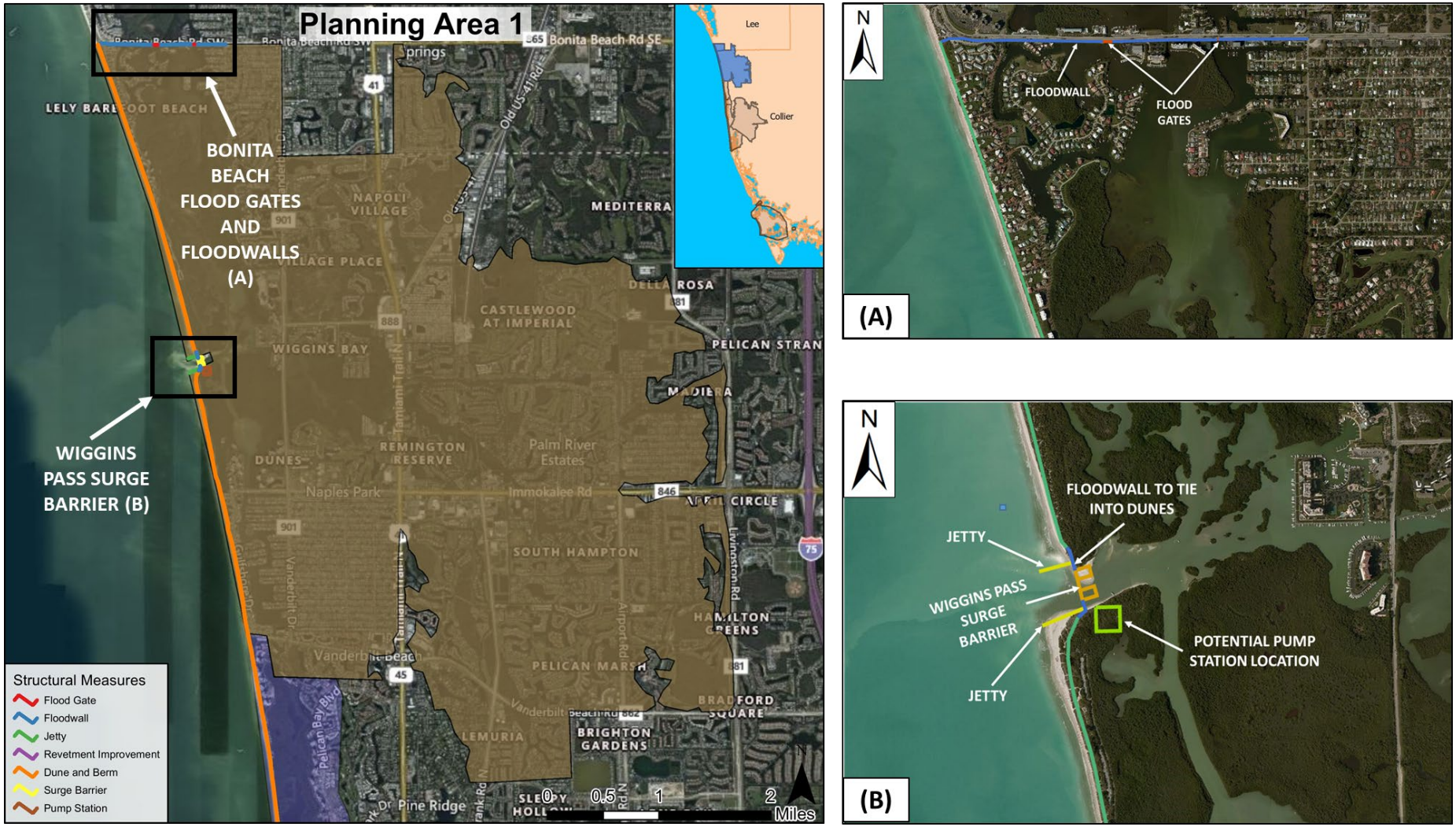


Figure 5.1 Planning Area 1 Alignment

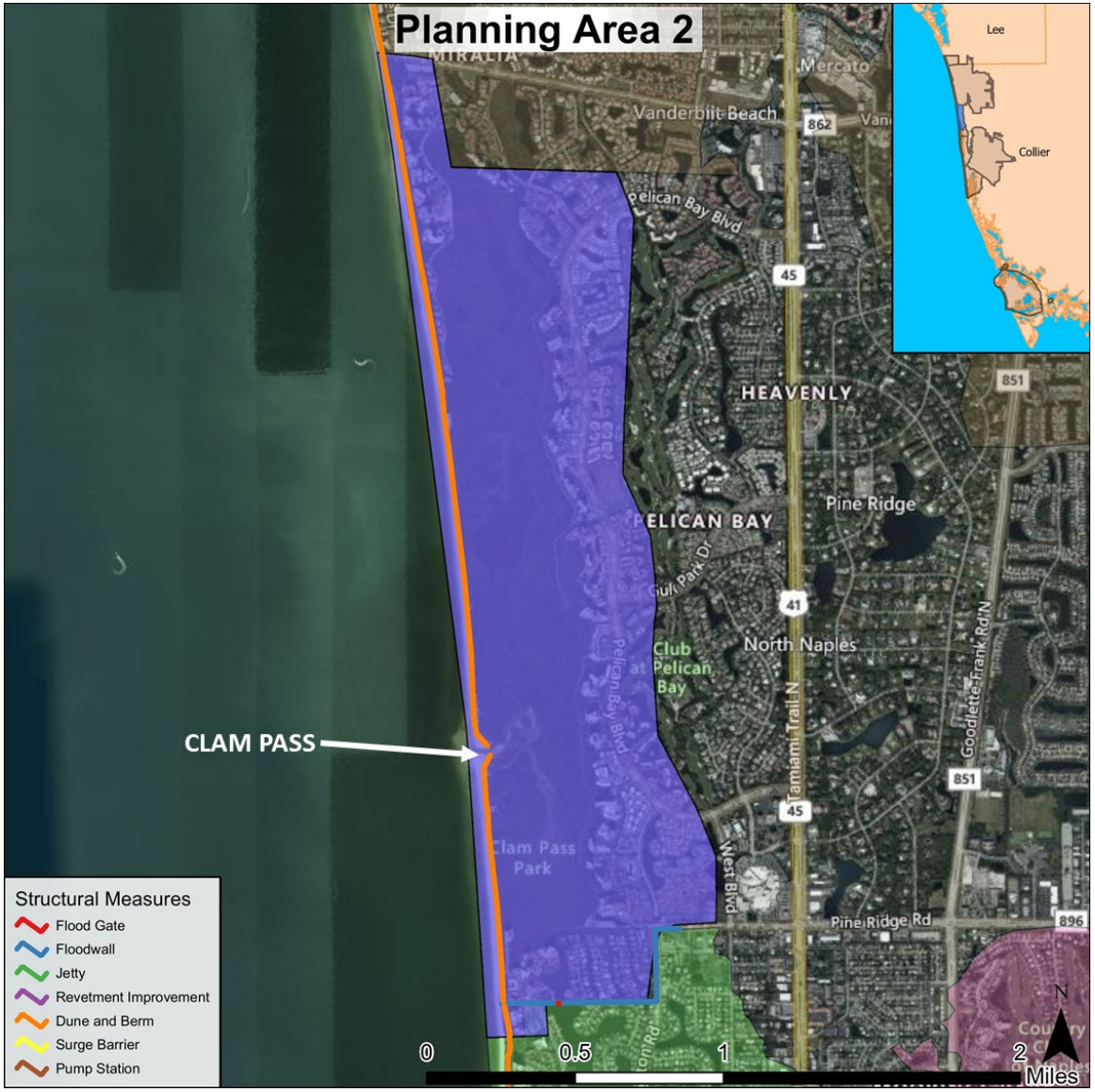


Figure 5.2 Planning Area 2 Alignment

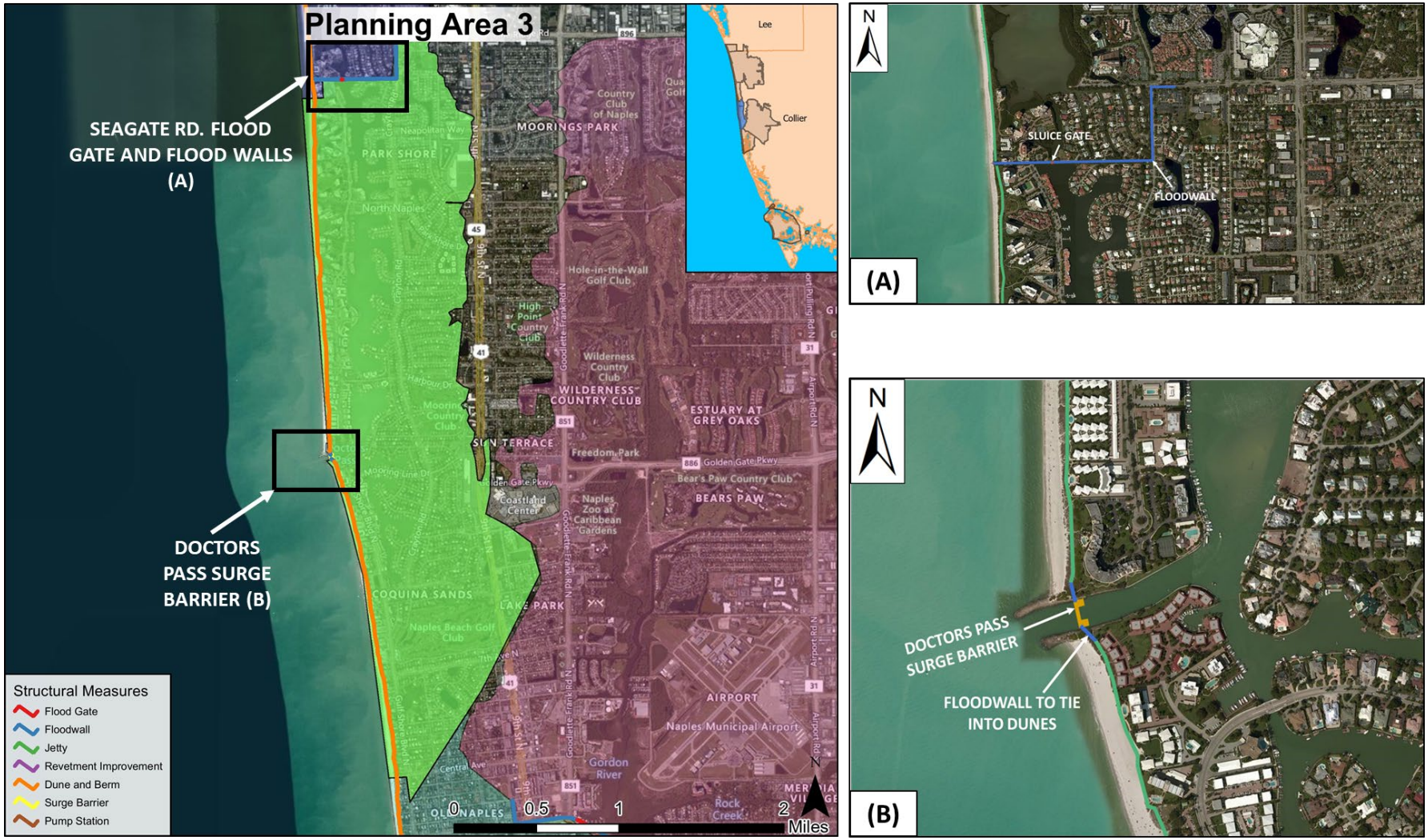


Figure 5.3 Planning Area 3 Alignment

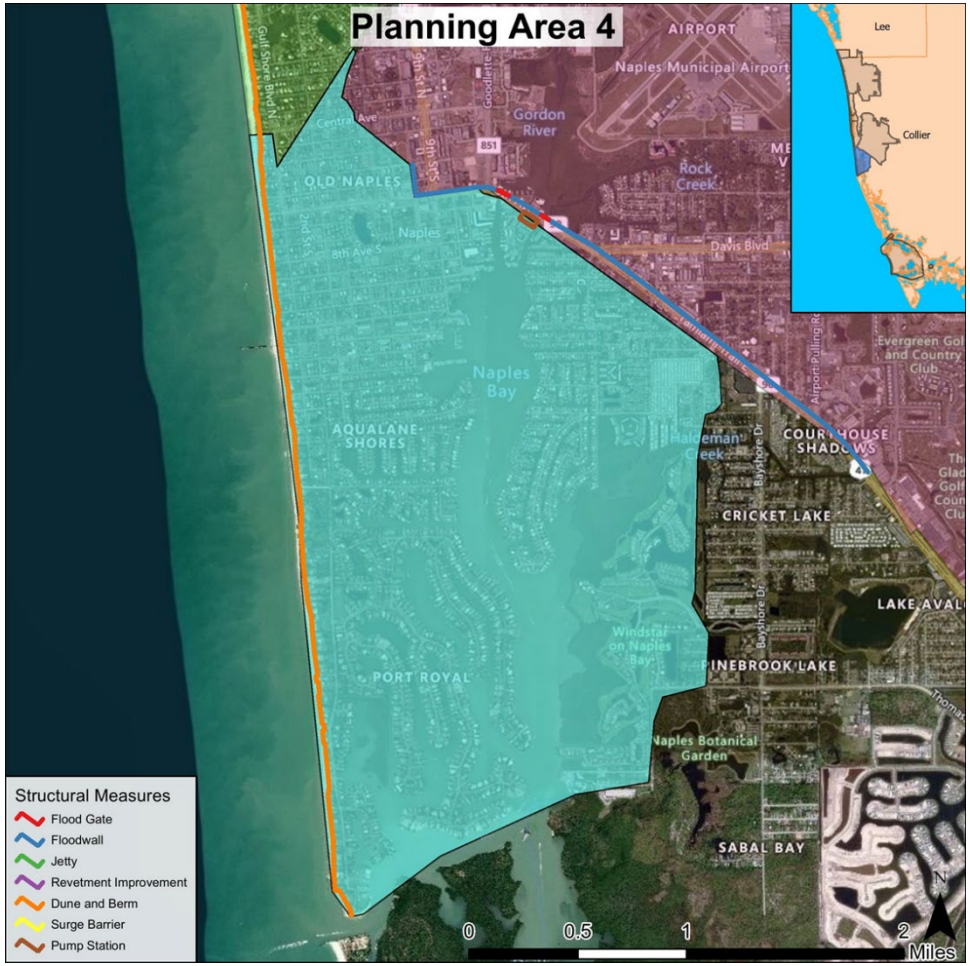


Figure 5.4 Planning Area 4 Alignment

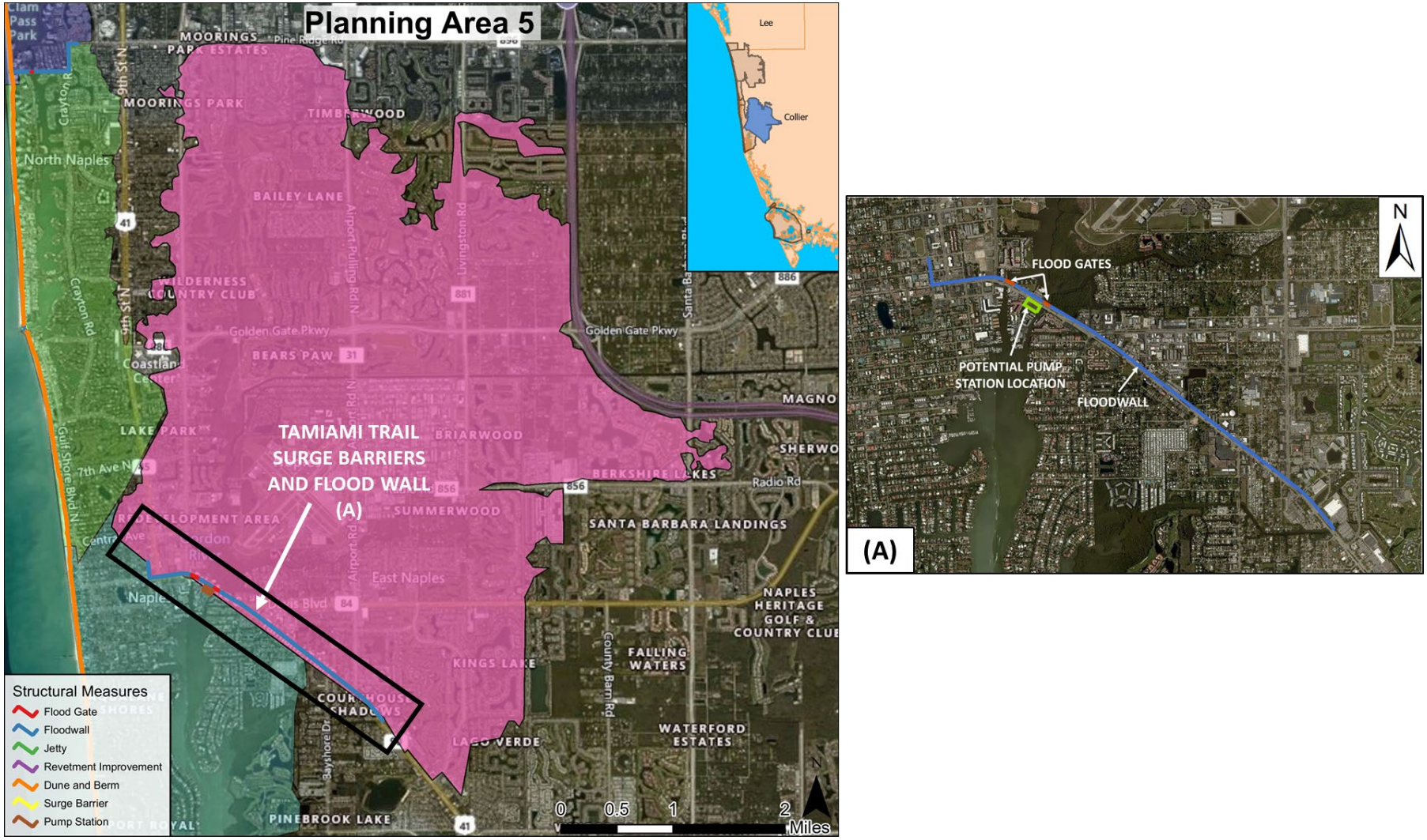


Figure 5.5 Planning Area 5 Alignment

Figure 5.6 Planning Area 6 Alignment (Marco Island)

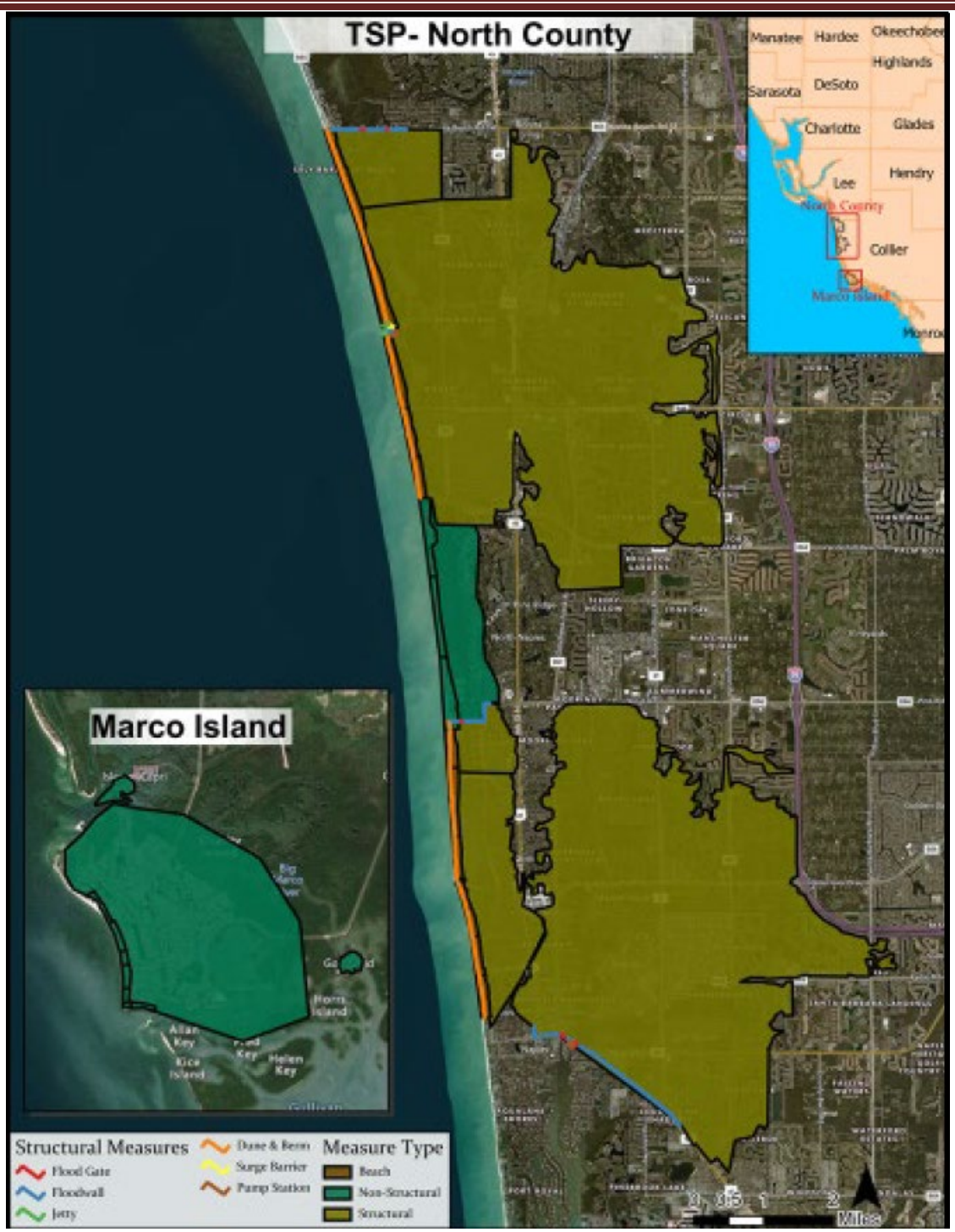


Figure 5.7 TSP Overview

5.3. PROJECT STILL WATER LEVELS

Water levels were evaluated at several "Save Points" and along all the Collier County Beach Reaches (Figure 5.8). Table 5.1 and Table 5.2 show the FEMA SWFL Study Stillwater levels projected for 2030 and 2079 (which include RSLR) for different save points and beach reaches.

Note that Table 5.1 is not all the available save points, but a selection of a few to show the general distribution of project SWL frequencies throughout the project area. The tables show five frequencies (20-yr ARI flood, 50-yr ARI flood, 100-yr ARI flood, 200-yr ARI flood, and 500-yr ARI flood) which contributed to the engineering analysis for shoreline protection. The PDT determined that the 2079 20-yr flood (50%) event was the minimum stillwater elevation that would create substantial enough flood damages to justify a large flood reduction construction project as the heights are approximately the same or less in many areas along the beach as the existing dune heights, an thus is the lowest limit in the table for analysis (See 5.4.1.4 COASTAL MORPHOLOGY for more information on the existing beach profiles). The table assumes relative sea level rise to the years 2030 and 2079 based on the USACE intermediate curve. The PDT decided that the 100-yr ARI flood (1%) stillwater elevation should be the primary value used in the range of elevations used for formulating separable measures throughout Collier County, with potential in increase to 200-yr and 500-yr levels if existing topography to tie in and economics allow. One can see in Table 5.2 that the SWL generally decrease from North to South along the beaches. To simplify plan formulation and model runs, individual SWL at each area were not considered for the heights of structures, but instead the overall range of SWL at the 100-yr, 200-yr, and 500-yr events (both in the year 2030 and 2079) were evaluated to simplify to the elevations of proposed structures to 10 FT, 12 FT, and 14 FT across the board. These were set also set by considering that the general low elevation in Collier County limits the ability to tie into ground much higher than 12-14 ft. Existing beach dunes were also evaluated at their existing heights which as described above are approximately the same height as a 20-yr event.

Water Levels and the projected rates of Sea Level Rise are discussed in more detail in the Hydraulics, Hydrology, and Coastal Sub-Appendix at the end of this Engineering Appendix.

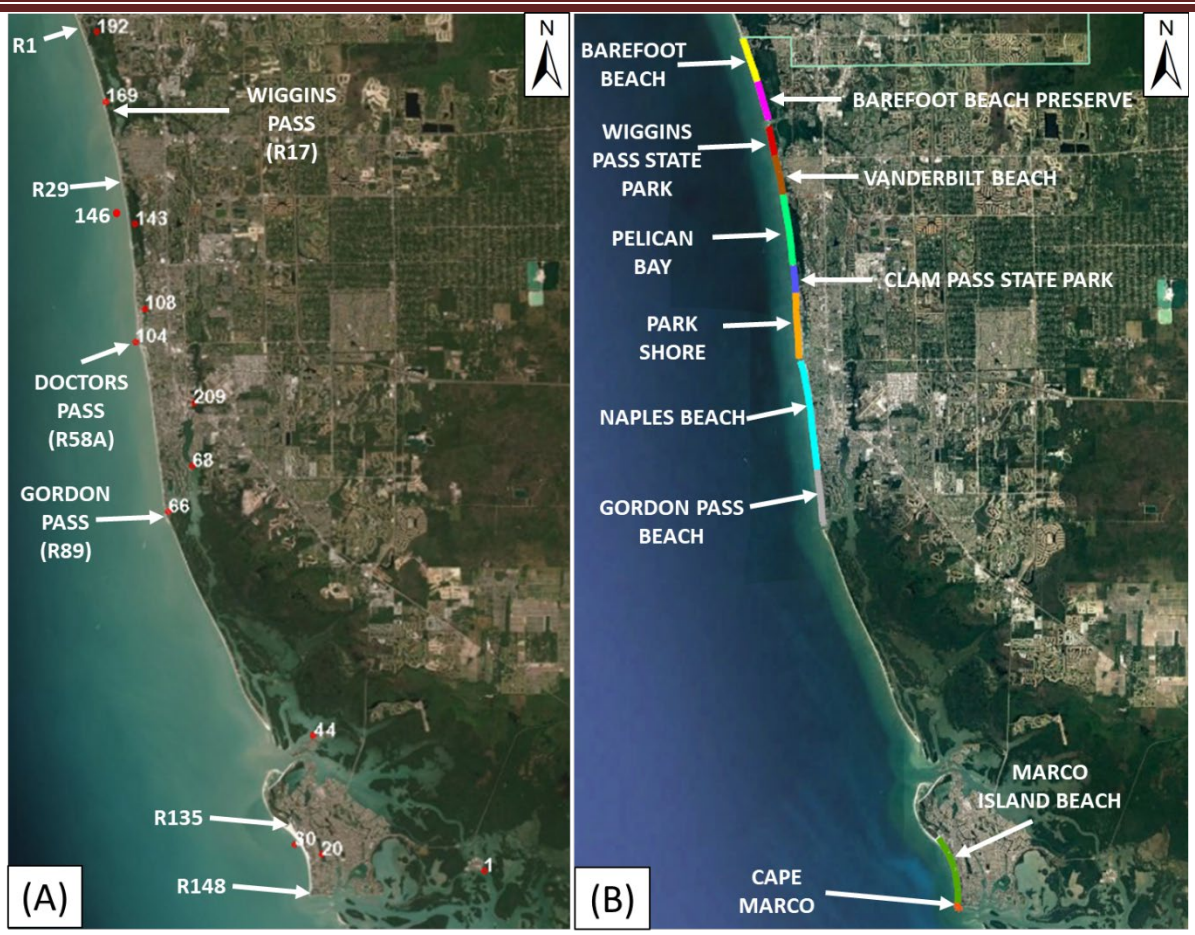


Figure 5.8. (A) Collier County Save Point and R-Monument Locations (B) Collier County Beach Reaches

Table 5.1 Water Levels (in feet) for selected Collier County Save Points at 2030 and 2079

		Study Surge Elevations (FT NAVD88)-Year 2030					Study Surge Elevations (FT NAVD88)-Year 2079				
Save Point Number	**Area Description**	20-YR	50-YR	100-YR	200-YR	500-YR	20-YR	50-YR	100-YR	200-YR	500-YR
1	Goodland	7.4	9.4	11.1	12.7	14.6	8.4	10.4	12.1	13.7	15.7
20	Inside Marco Island	6.5	8.3	9.6	10.9	12.4	7.5	9.3	10.6	11.9	13.4
30	Offshore Marco Island	5.6	7.4	8.9	10.3	12.0	6.6	8.4	9.9	11.3	13.0
44	Isles of Capri	5.3	6.9	8.3	9.6	11.2	6.3	7.9	9.3	10.6	12.2
66	Gordon Pass Inlet	5.4	7.0	8.2	9.6	11.9	6.4	8.0	9.2	10.6	13.0
68	Inside Gordon Pass (South)	5.5	7.0	8.3	9.6	11.5	6.5	8.0	9.3	10.6	12.5
104	Doctors Pass Inlet	5.8	7.9	9.2	10.6	13.2	6.8	8.9	10.3	11.6	14.2
108	Inside Doctors Pass (North)	6.4	8.5	9.8	11.3	14.3	7.4	9.5	10.9	12.4	15.3
143	Inside Clam Pass	5.0	7.8	10.2	11.9	14.9	6.0	8.8	11.2	13.0	15.9
146	Offshore North County Beaches	6.2	8.5	10.0	11.7	14.4	7.2	9.5	11.1	12.7	15.4
169	Wiggins Pass Inlet	6.1	8.2	9.8	11.6	14.8	7.1	9.2	10.9	12.6	15.8
192	Bonita Beach Rd Floodwall	6.0	7.8	9.4	11.4	14.9	7.0	8.8	10.4	12.4	15.9
209	Tamaimi Trail Floodwall	5.8	7.4	8.6	9.8	11.5	6.8	8.4	9.6	10.8	12.5

Table 5.2 Water Levels (in feet) for Collier County Beach Reaches at 2030 and 2079

		Study Surge Elevations (FT NAVD88)-Year 2030					Study Surge Elevations (FT NAVD88)-Year 2079				
	Reach	20-YR	50-YR	100-YR	200-YR	500-YR	20-YR	50-YR	100-YR	200-YR	500-YR
1	Barefoot Beach (R1-R9)	3.7	7.8	10.3	12.2	15.1	4.7	8.8	11.3	13.2	16.1
2	Barefoot Beach Preserve (R9-R16)	5.8	8.1	10.0	11.9	15.0	6.8	9.1	11.0	12.9	16.0
3	Wiggins Pass State Park (R17-R22)	6.0	8.2	9.8	11.7	14.8	7.0	9.2	10.8	12.7	15.8
4	Vanderbilt Beach (R22-R29)	5.8	8.1	9.7	11.5	14.7	6.8	9.1	10.7	12.5	15.7
5	Pelican Bay (R29- R41)	5.9	8.2	9.8	11.5	14.4	6.9	9.2	10.8	12.5	15.4
6	Clam Pass State Park (R42-R46)	6.0	8.3	9.8	11.4	14.1	7.0	9.3	10.8	12.4	15.1
7	Park Shore (R46- R57)	4.7	8.0	9.5	11.0	13.7	5.7	9.0	10.5	12.0	14.7
8	Naples Beach (R58A - R79)	3.7	6.6	8.4	10.2	12.9	4.8	7.6	9.5	11.2	13.9
9	Gordon Pass Reach (R79-R89)	4.9	6.6	8.1	9.8	12.1	5.9	7.7	9.1	10.8	13.1
10	Marco Island Beach (R135-R148)	4.6	6.9	8.9	10.2	11.9	5.6	7.9	9.9	11.3	12.9
11	Cape Marco (R148- G-4)	5.5	7.3	8.7	10.1	11.6	6.5	8.3	9.7	11.1	12.6

5.4. BEACH MEASURES ANALYSIS

The following paragraphs summarize the engineering evaluation and modeling performed to analyze the beach "structural" measures (dune and berm systems) along the Collier County North County beaches and Marco Island.

5.4.1. BEACH-FX LIFE-CYCLE SHORE PROTECTION PROJECT EVOLUTION MODEL ANALYSIS

Federal participation in Coastal Storm Risk Reduction (CSRM) projects is based on a favorable economic justification in which the benefits of the project outweigh the costs. Determining the Benefit to Cost Ratio (BCR) requires both engineering (project performance and evolution) and planning (alternative analysis and economic justification) analyses. The interdependence of these functions has led to the development of the life-cycle simulation model Beach-*fx*. Beach-*fx* combines the evaluation of physical performance and economic benefits and costs of shore protection projects (Gravens et. al., 2007), particularly beach nourishment, to form the basis for determining the justification for Federal participation. This section describes the engineering aspects of the Beach-*fx* model.

5.4.1.1. BACKGROUND AND THEORY

Beach-*fx* is an event-driven life-cycle model. USACE guidance (USACE, 2006) requires that flood damage reduction studies include risk and uncertainty. The Beach-*fx* model satisfies this requirement by fully incorporating risk and uncertainty throughout the modeling process (input, methodologies, and output). Over the project life-cycle, typically 50 years, the model estimates shoreline response to a series of storm events, for this study a synthetic storm suite that is discussed further in 5.4.1.3 METEROLOGICAL DRIVING FORCES. These plausible storms, the driving events, are randomly generated using a Monte Carlo simulation. The corresponding shoreline evolution includes not only erosion due to the storms, but also allows for storm recovery, post-storm emergency dune and/or shore construction, and planned nourishment events throughout the life of the project. Risk based damages to structures are estimated based on the shoreline response in combination with pre-determined storm damage functions for all structure

types within the project area. Uncertainty is incorporated not only within the input data (storm occurrence and intensity, structural parameters, structure and contents valuations, and damage functions), but also in the applied methodologies (probabilistic seasonal storm generation and multiple iteration, life cycle analysis). By using a storm suite that is sampled randomly based on relative and seasonal probabilities, the uncertainty of occurrence of any give storm, regardless of intensity is assured through the Monte Carlo sampling scheme as well as the multiple iterations of the project lifecycle. Results from multiple iterations of the life cycle can be averaged or presented as a range of possible values. For more general information on Beach-*fx,* please refer to the Beach-*fx* User's Manual (ERDC/CHL SR-09-6) and article "Beach-fx: Monte Carlo Life-Cycle Simulation Model for Estimating Shore Protection Project Evolution and Cost Benefit Analyses" (Gravens et. al., 2007).

5.4.1.2. MODEL REACH SETUP

The project site itself is represented by divisions of the shoreline referred to as "Reaches". Because this term may also be used to describe segments of the shoreline to which project scenarios are applied, Beach-*fx* reaches will be referred to in this appendix as "Model reaches". Model reaches are contiguous, morphologically homogenous areas that contain groupings of structures (residences, businesses, walkovers, roads, etc…), all of which are represented by Damage Elements (DEs). DEs are grouped within divisions referred to as Lots. Figure 5.9 shows a graphic depiction of the model setup. For further details about the specifics of Lot extents and DE grouping see the Economics Appendix.

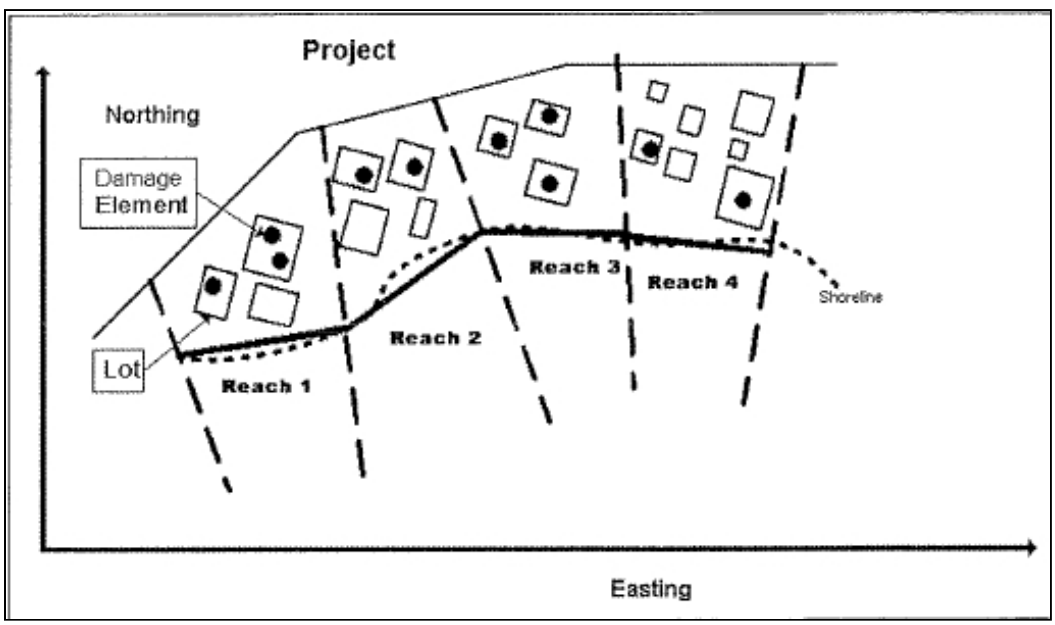


Figure 5.9 Beach-*fx* Model Setup Representation

Each model reach is associated with a representative beach profile that describes the cross-shore profile of the reach. While an effort is made to designate model reaches to include a single

DNR monument (FDEP R-monument) from which historical survey data can be used to establish a representative profile for that reach, the positioning of the monument within each reach and the length of each reach are variable. Multiple model reaches may share the same representative beach profile and groupings of model reaches may represent a single design reach. For Collier County, the North County the project area consists of 22 model reaches and Marco Island consists of 4 model reaches. Table 5.3 provides model reach identifiers as well as the FDEP R-monument that falls within the borders of each model reach. The tables also identifies the Planning Area associated with each used for purpose of grouping areas for economic analysis and beach reach name. The reaches are also shown in Figure 5.10 to Figure 5.16 with the Beach-*fx* damages elements in each reach. See the Economics Appendix for more information on the development of the damage areas.

Table 5.3 Collier County Beach-*fx* Reaches

North County Beaches				Marco Island Beaches			
Planning Area	**Beach Reach**	**Model Reaches**	**R-monuments**	**Planning Area**	**Beach Reach**	**Model Reaches**	**R-monuments**
1	Barefoot Beach	BB1	R-1 to R-9	6	Marco Island	MI1	R-135 to R-138
	Barefoot Beach Preserve	BB2	R-9 to R-16			MI2a	R-138 to R-141
	Wiggins Pass State Park	WP	R-17 to R-22			MI2c	R-141 - 142
	Vanderbilt	VB1	R-22 to R-26			MI2b	R-142 to R-143.5
		VB2	R-26 to R-29			MI3	R-143.5 to R-146.5
2	Pelican Bay	PB1	R-29 to R-35			MI4a	R-146.5 to R-148
		PB2	R-35 to R-41		Cape Marco	MI4b	R-148 to G-4
	Clam Pass State Park	CP	R-42 to R-46				
3	Park Shore	PS1	R-46 to R-49				
		PS2a	R-49 to R-54				
		PS2b	R-54 to R-57				
	Naples	NB1	R-58A to R-59				
		NB2	R-59 to R-61				
		NB3	R-61 to R-64				
		NB4	R-64 to R-68				
4	Naples	NB5	R-68 to R-70				
		NB6	R-70 to R-76				
		NB7	R-76 to R-79				
	Gordon Pass	GP1	R-79 to R-83				
		GP2	R-83 to R-87				
		GP4	R-87 to R-88.5				
		GP3	R-88.5 to R-89				

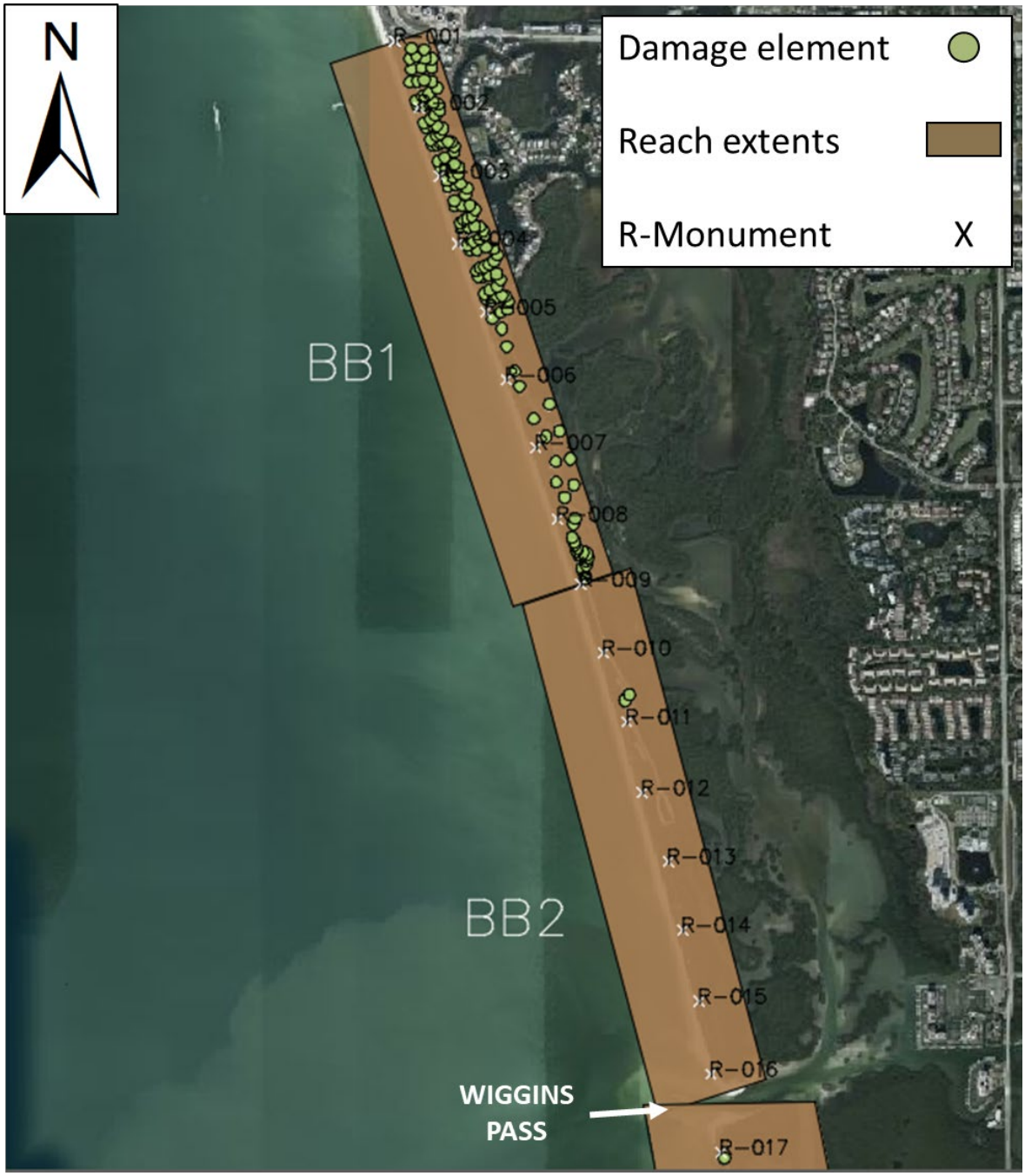


Figure 5.10 North County Beach-*fx* Model Reaches BB1 to BB2

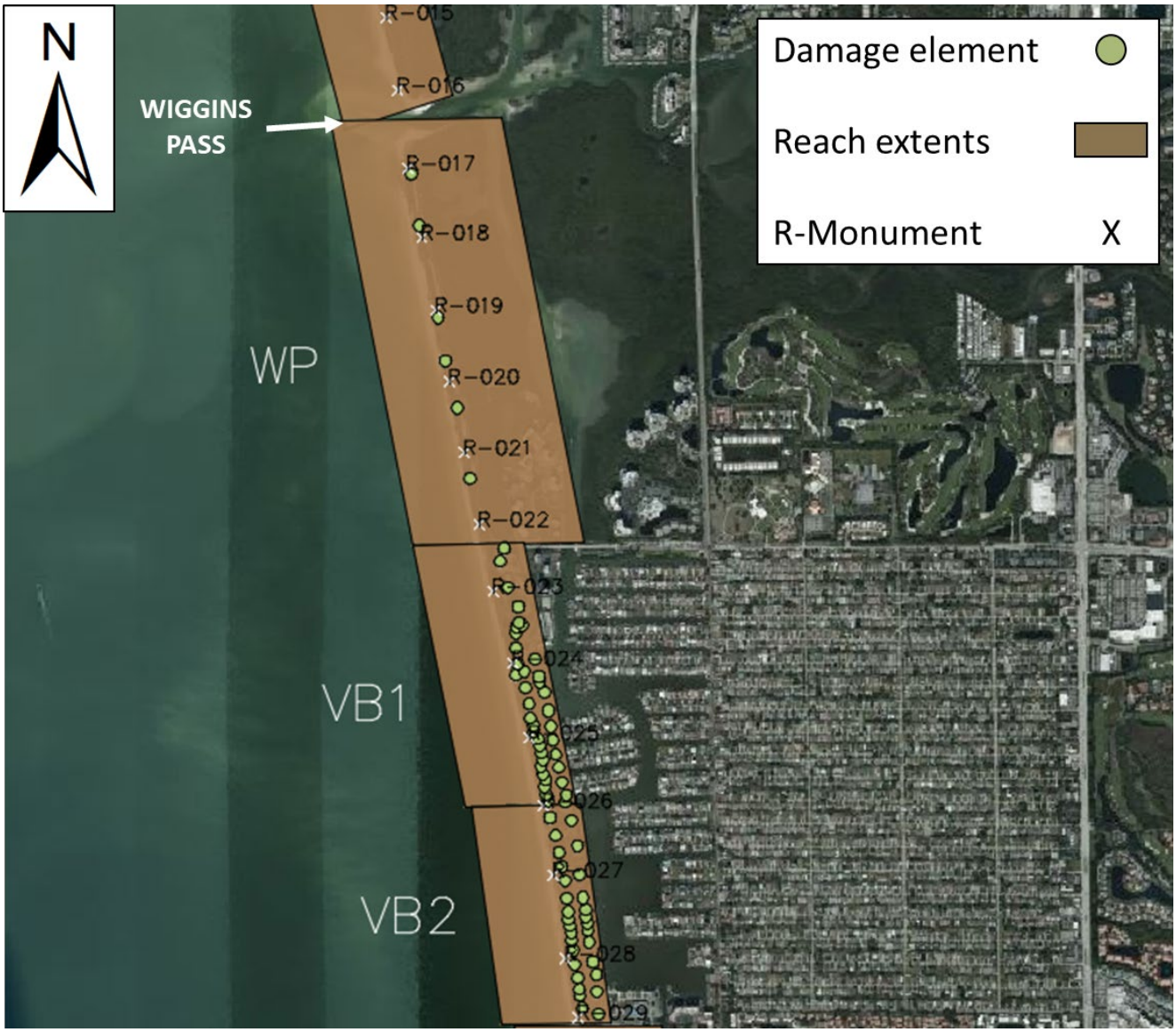


Figure 5.11 North County Beach-*fx* Model Reaches WP to VB2

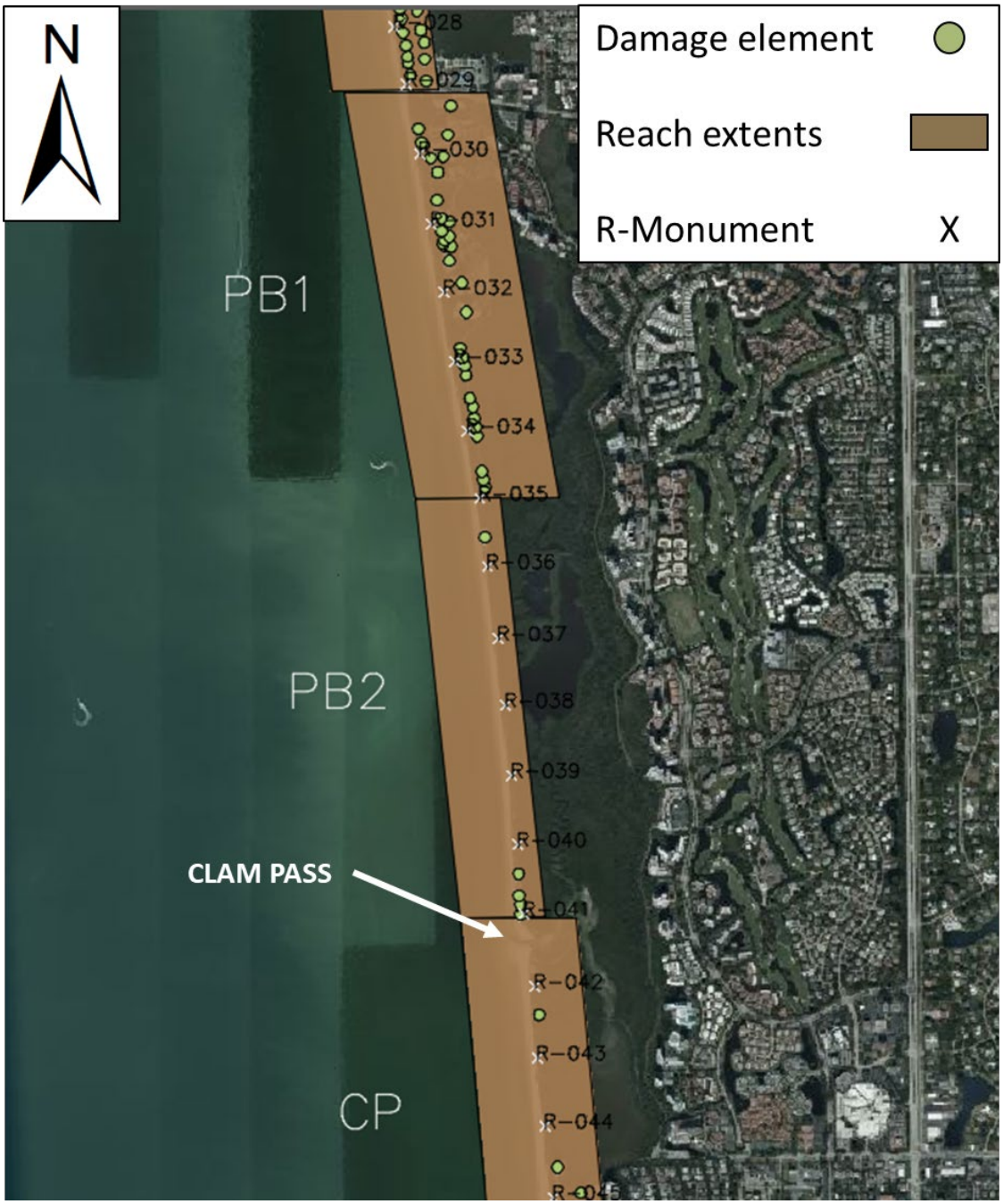


Figure 5.12 North County Beach-*fx* Model Reaches PB1 to CP

Figure 5.13 North County Beach-*fx* Model Reaches CP to PS2b

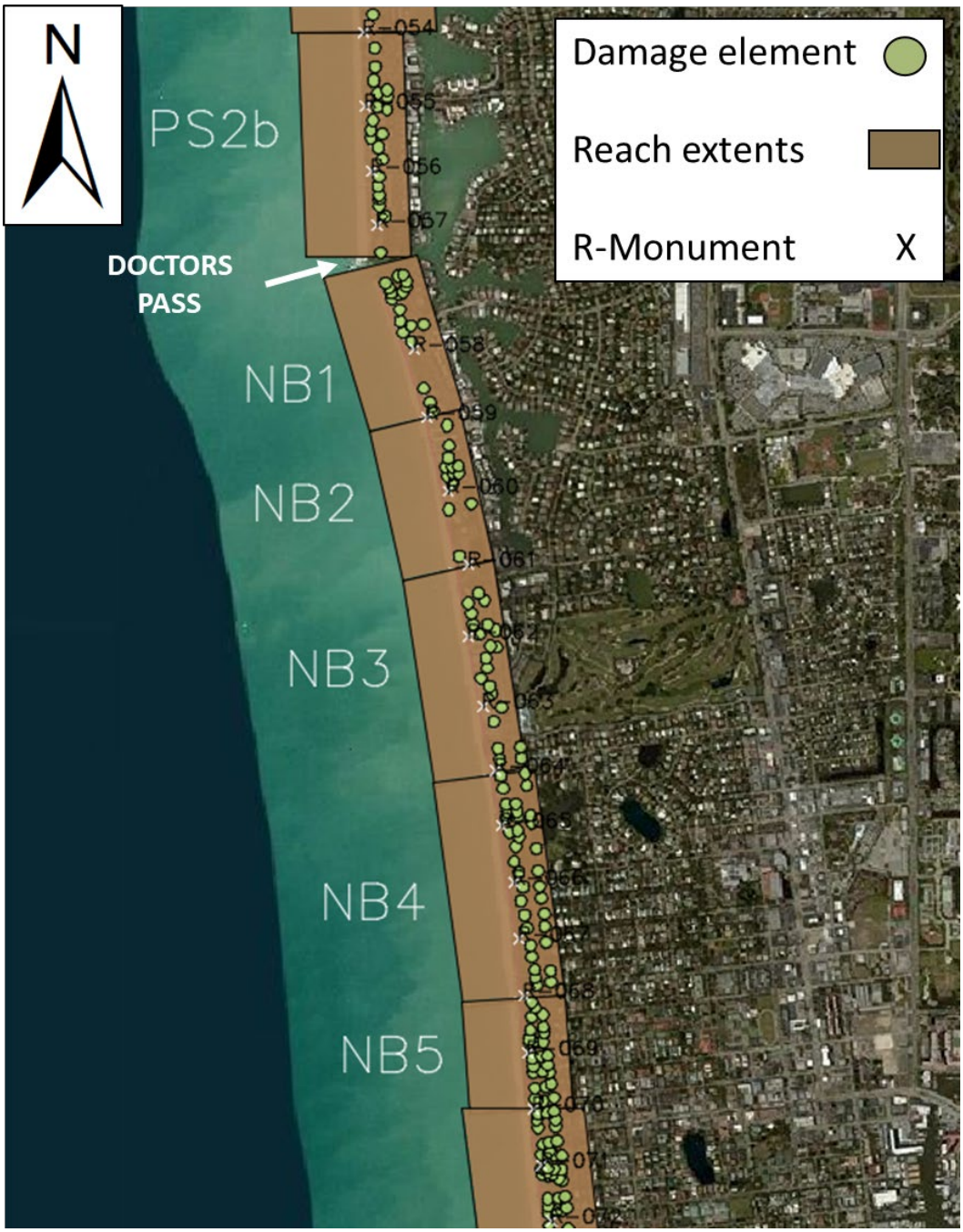


Figure 5.14 North County Beach-*fx* Model Reaches PS2b to NB5

Figure 5.15 North County Beach-*fx* Model Reaches NB6 to GP3

Figure 5.16 Marco Island Beach-*fx* Beach Reaches

Implementation of the Beach-*fx* model relies on a combination of meteorology, coastal engineering, and economic analyses and is comprised of four basic elements:

- Meteorological driving forces
- Coastal morphology
- Economic evaluation
- Management measures

The subsequent discussion in this section addresses the basic aspects of implementing the Beach-fx model. For a more detailed description of theory, assumptions, data input/output, and model implementation, refer to Gravens et al. (2007), Males et al. (2007), and USACE (2009).

5.4.1.3. METEROLOGICAL DRIVING FORCES

The predominant driving force for coastal morphology and associated damages within the

Beach-*fx* model is the historically based set of storms that is applied to the life-cycle simulation. The predominant driving force for erosion, inundation, and wave damages on western coast of Florida is tropical storms (hurricanes) in the summer months. Extra-tropical storms are mild compared to east coast Northeasters and do not significantly impact morphological changes beyond their contribution to the background erosion rate.

Representative Storm Suites

As discussed in the Hydraulics, Hydrology, & Coastal Sub-Appendix, the 357 storms were provided as part of the processed data provided from the FEMA SWFL Study for each save point. For Collier County, the storms for two of these save points were analyzed, one for the North County Beaches and another for the Marco Island Beaches. These point locations are shown in Figure 5.17.

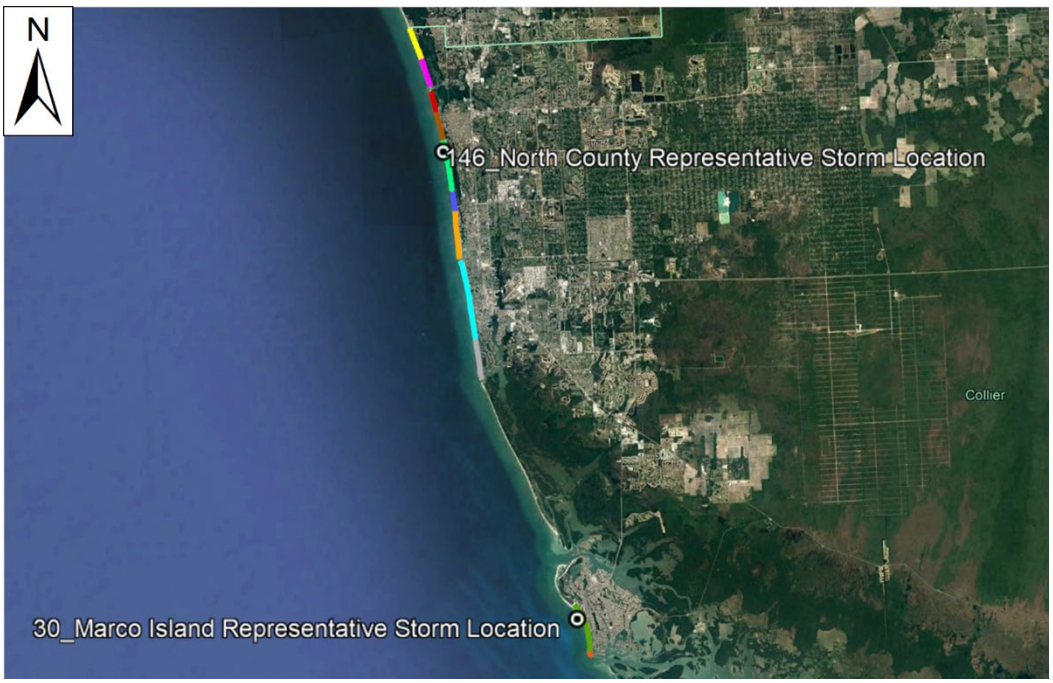


Figure 5.17 Collier County save point locations used to create storm suite for Beach-*fx* storms

To reduce the amount of storms and increase computation efficiency, a smaller number of representative storms was selected at each point. The process to identify and select the representative storms followed the following step (Gravens et al., 2018):

- Group storms into clusters based on the magnitude of the peak surge generated
- Further sub-divide storm clusters if appropriate, based on the duration of storm surge hydrographs or low versus high-peak amplitude.
- Select representative storm events within each storm cluster
- Compute the relative probability of the representative storm

The number of clusters and specific peak storm surge limits defining each cluster are arbitrary, and in this procedure the choice was made to use the available AEP to define the clusters. Each of the 357 storms provided for Collier County was assigned to one of the storm clusters by determining the peak surge elevation of the storms and assigning it to the cluster with upper and lower limits bounding the peak. The lower and upper limits of the storm clusters are computed as the midpoint value between the current storm cluster stage value and the adjacent (lower and upper) storm cluster stage values. Table 5.4 lists the boundaries of the storm clusters for the point locations shown in Figure 5.17. Note that the first storm cluster is assigned 0.3 m (~ 1 ft) for each point as there is no storm which produces a peak less than this value. Also note that the hydrographs which were evaluated were clusters were the original hydrographs not adjusted for SLR. All necessary SLR adjustments were made to the hydrographs post representative storm analysis.

Table 5.4 Lower and upper stage limits for selecting Beach-*fx* representative storms

North County Save Point 146 Lower and Upper stage Limits			Marco Island Save Point 30 Lower and Upper stage Limits		
Lower Stage (m, MSL)	Return Year/Cluster Number	Upper Stage (m, MSL)	Lower Stage (m, MSL)	Return Year/Cluster Number	Upper Stage (m, MSL)
0.30	5/1	1.00	0.30	5/1	1.23
1.00	10/2	1.54	1.23	10/2	1.63
1.54	20/3	2.05	1.63	20/3	2.01
2.05	50/4	2.76	2.01	50/4	2.57
2.76	100/5	3.25	2.57	100/5	3.02
3.25	200/6	3.75	3.02	200/6	3.45
3.75	500/7	4.56	3.45	500/7	3.96
4.56	100/8		3.96	100/8	

After the storms have been assigned to the storm clusters, the storm surge hydrographs time series of all storms in each cluster are translated along the time axis such that the peak surge is aligned at a single times. Based on visual inspections of the aligned storm hydrographs, the storms in the clusters were further sub-divided into sub clusters to capture different storm surge hydrograph characteristics present in the cluster, i.e. to separate storms of long durations from storms with shorter duration. Clusters were also further subdivided by peak elevation if the original cluster had a large number of storms. Figure 5.18 and Figure 5.19 show examples of how clusters were visually inspected and representative storms selected for storms at Save Point 146.

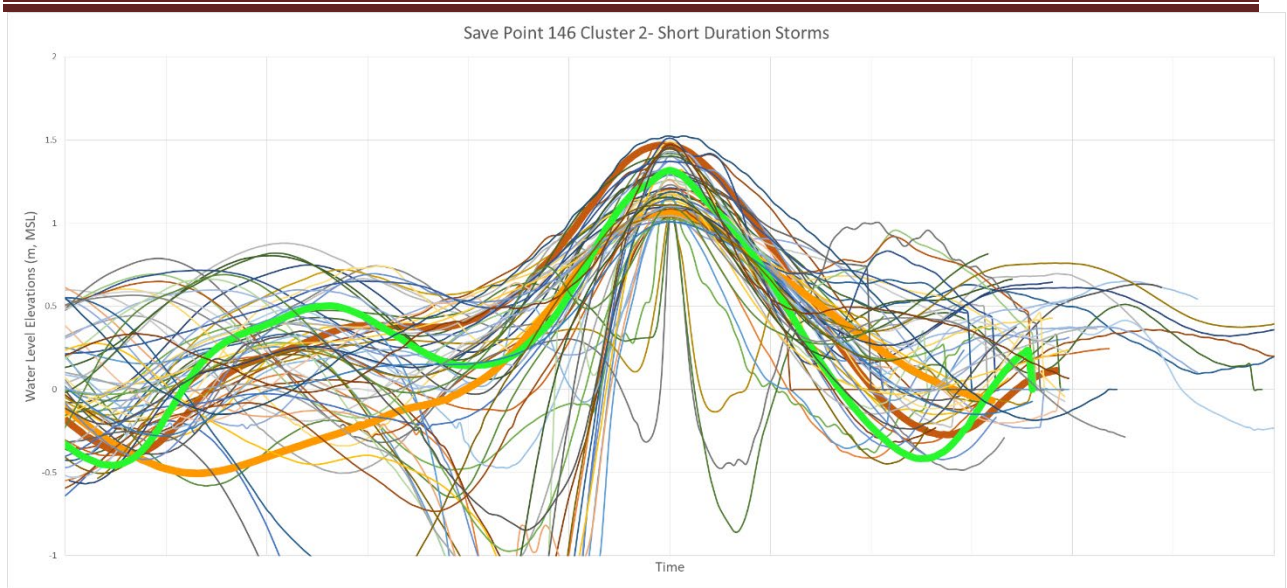


Figure 5.18 Selected Representative Storms for Save Point 146 Cluster 2 (10 YR) Short duration storms

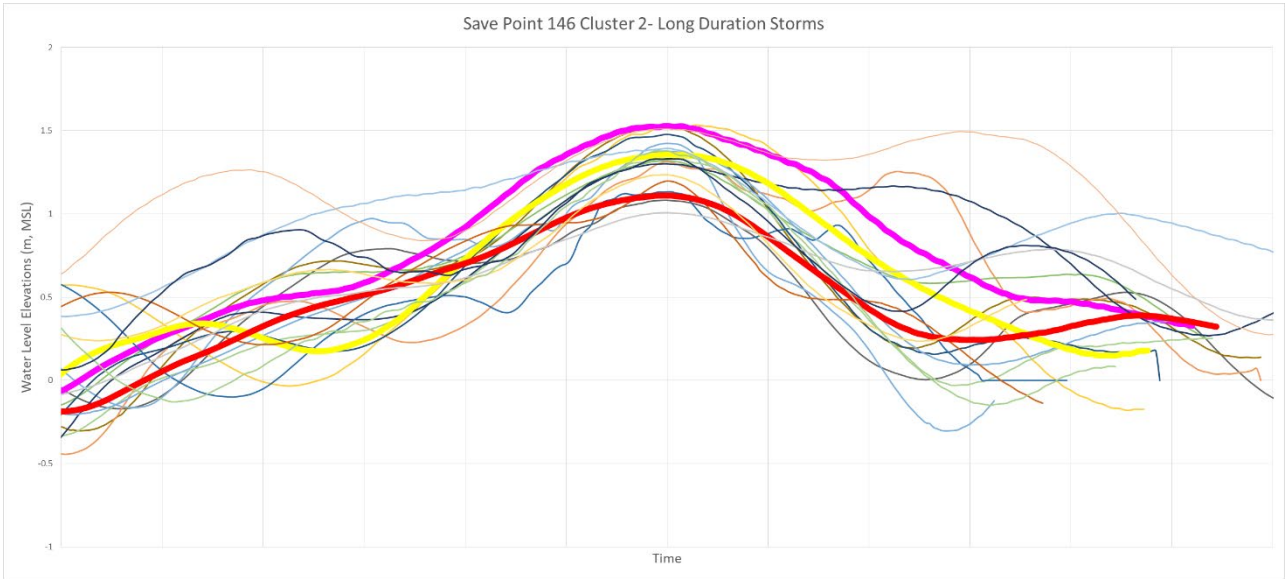


Figure 5.19 Selected Representative Storms for Save Point 146 Cluster 2 (10 YR) long-duration storms

Selection of representative storms or storms within the storm clusters using the methodology outlined herein is subjective in that the analyst ultimately selects a representative storm or storms from each of the developed storm clusters based on judgment of which hydrograph best characterizes the collection of hydrographs in the cluster. The final step is the procedure is the calculation of the relative probability to be assigned to each of the representative storms. For the FEMA SWFL Storms, the data provided include storm rate with units storms/year. For Beach-fx, the relative probability (i.e. relative weight) is required, which was calculated by normalizing the

provided storm rate values by dividing all of the individual storm rates by the largest storm rate in the storm suite. The result of this normalization process is the generation of a non-dimensional quantity that reflects the relative probability or relative weight of the each storm relative to all other storms in the storm suite. Therefore, the individual storm relative probabilities range in value from a minimum of 0 to a maximum of 1. The relative probability of a selected representative storm is computed by summing the relative probabilities of all the storms in the storm cluster that storm represents. Table 5.5 and Table 5.6 list the selected representative storms, the computed relative probabilities, and the number of storms represented for each of the storm clusters listed in Table 5.4.

Table 5.5 Selected North County representative tropical storms and relative probabilities for save point 146

Cluster Number	Representative Storm Number	Relative Probability	Number of Storms Represented	Note
1	Synthetic_89	2.586	18	1
1	Synthetic_125	3.832	19	4
1	Synthetic_335	8.684	53	12
1	Synthetic_305	5.353	24	13
1	Synthetic_266	2.410	24	14
1	Synthetic_81	0.468	10	7
1	Synthetic_301	4.851	26	3
2	Synthetic_225	4.353	28	6
2	Synthetic_282	3.612	34	7
2	Synthetic_128	0.326	6	8
2	Synthetic_293	1.295	4	9
2	Synthetic_263	2.863	13	10
2	Synthetic_84	0.806	5	11
3	Synthetic_103	0.751	9	7
3	Synthetic_87	1.017	6	14
3	Synthetic_323	0.998	8	12
3	Synthetic_303	0.562	8	10
4	Synthetic_210	0.365	4	8
4	Synthetic_156	0.413	5	6
4	Synthetic_119	0.208	8	13
4	Synthetic_170	1.653	18	10
5	Synthetic_330	0.090	5	10
5	Synthetic_79	0.416	3	7
6	Synthetic_67	0.388	10	2
7	Synthetic_164	0.063	1	2
8	Synthetic_192	0.104	7	7
8	Synthetic_350	0.002	1	10

Notes:

1	Low amplitude storms
2	mean amplitude storms
3	high amplitude storms
4	long-duration storms
5	short duration storms
6	short-duration, low amplitude storms
7	short-duration,mean amplitude storms
8	short-duration,high amplitude storms
9	Long-duration, low amplitude storms
10	Long-duration,mean amplitude storms
11	Long-duration,high amplitude storms
12	Mean-duration, low amplitude storms
13	Mean-duration, mean amplitude storms
14	Mean-duration, high amplitude storms

Table 5.6 Selected Marco Island representative tropical storms and relative probabilities for save point 30

Cluster Number	Representative Storm Number	Relative Probability	Number of Storms Represented	Note
1	Synthetic_42	0.267	5	2
1	Synthetic_89	5.528	28	2
1	Synthetic_335	2.380	24	10
1	Synthetic_345	1.558	12	13
1	Synthetic_354	6.743	35	10
1	Synthetic_318	1.615	9	13
1	Synthetic_50	3.481	29	12
1	Synthetic_355	2.558	10	9
1	Synthetic_9	1.026	8	13
1	Synthetic_274	2.454	15	10
1	Synthetic_22	1.020	10	8
1	Synthetic_255	1.719	16	14
1	Synthetic_293	0.688	7	11
2	Synthetic_128	3.416	21	12
2	Synthetic_138	0.778	6	9
2	Synthetic_146	0.478	7	7
2	Synthetic_137	1.300	11	13
2	Synthetic_263	1.148	5	10
2	Synthetic_145	1.178	9	14
2	Synthetic_110	0.381	4	11
3	Synthetic_178	1.232	6	6
3	Synthetic_119	0.305	8	8
3	Synthetic_325	0.312	7	12
3	Synthetic_77	1.613	9	14
3	Synthetic_342	0.052	4	4
4	Synthetic_213	0.730	7	7
4	Synthetic_160	0.632	8	13
4	Synthetic_86	0.288	5	10

Cluster Number	Representative Storm Number	Relative Probability	Number of Storms Represented	Note
5	Synthetic_183	0.065	2	7
5	Synthetic_161	0.513	9	13
5	Synthetic_330	0.066	1	10
6	Synthetic_220	0.127	4	7
6	Synthetic_33	0.119	6	13
6	Synthetic_331	0.069	2	10
7	Synthetic_70	0.024	3	13
7	Synthetic_350	0.002	1	10
8	Synthetic_194	0.023	3	13
8	Synthetic_351	0.002	1	10

Notes:

1	Low amplitude storms
2	mean amplitude storms
3	high amplitude storms
4	long-duration storms
5	short duration storms
6	short-duration, low amplitude storms
7	short-duration,mean amplitude storms
8	short-duration,high amplitude storms
9	Long-duration, low amplitude storms
10	Long-duration,mean amplitude storms
11	Long-duration,high amplitude storms
12	Mean-duration, low amplitude storms
13	Mean-duration, mean amplitude storms
14	Mean-duration, high amplitude storms

A total of 27 storms were selected as representative storms for the North County region and 38 representative storms for Marco Island. These storms are shown in Figure 5.20 and Figure 5.21. It should be noted that random phase tides were embedded in the data provided by FEMA and thus also included in the processed storm hydrographs provided from ERDC. The use of random phase tides in the FEMA data means that the peak water levels in the storms do not specifically match with high and low tidal events. Instead each storm, assigned a uniformly distributed random start time within the hurricane season, was simulated with a corresponding tide throughout the duration of the storm event based on local tide history as defined by tide predictions for the simulated hurricane season (FEMA 2017). Therefore, one cannot ensure that a storm hits during high tide (a worst case condition) or low tide (a best case condition). While this may limit the number of external tidal forcings available in the model as only one tidal event (the tide associated with the original ADCIRC run) is modeled per storm, the data was deemed sufficient to make planning level decisions for the project area. Future analysis during the PED phase should use updated storms with more tidal variation on TSP beaches reaches to refine and further develop the model.

As Extra-Tropical storms were not included in the FEMA SWFL Study storms because they are very minor storm events in this area of the Gulf of Mexico, Beach-*fx* is more stable with Extra-Tropical storms included in the storm suite. The Extra-Tropical storm suite used for Collier County was the same Extra-Tropical Storms as one developed for a previous project in Lee

County directly north of Collier County and deemed suitable for use in this study due to the proximity of the project (Lee County, Gasparilla Island Section 934 Study performed by USACE SAJ). For that study, 49 extra-tropical storms identified were identified from historical wave and water level records from WIS data covering 1996 to 2012. Typically extra-tropical storms are events (usually associated with Northeasters), which produce a wave height (H_s) greater than 2 meters. However, there were no events that reached that threshold in the WIS record (WIS station 73285) for Lee County. It was therefore decided that in the Gulf extra-tropical "storms" would have to be above average conditions lasting for a significant period of time. The criteria used was $H_s > 3$ feet lasting for at least 48 hours. An initial array of events were identified with WIS data with corresponding water elevations obtained from CO-OPS NOAA Station 8725110. Overlapping the WIS record with the available water level record from NOAA Station 8725110 resulted in coverage from 1996 to 2012. In that time frame 49 extra-tropical "storms" were identified. These storms are shown in Figure 5.22 as Collier County Extra-Tropical Storms. .

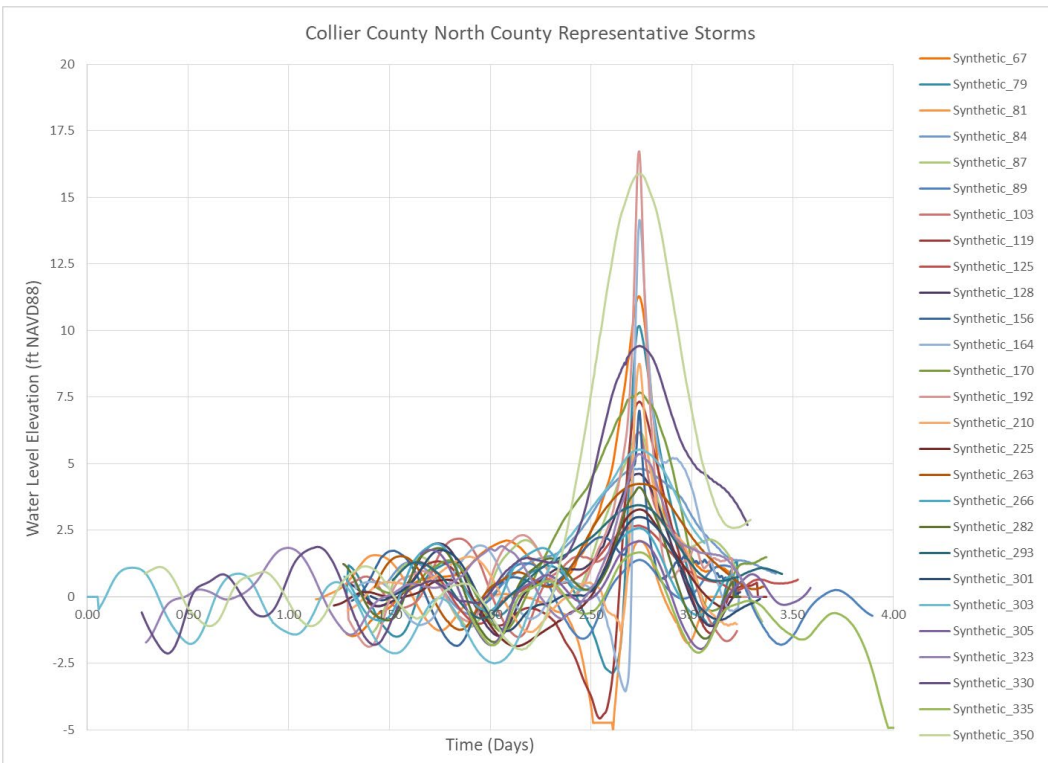


Figure 5.20 Collier County North County Representative Tropical Storm Hydrographs

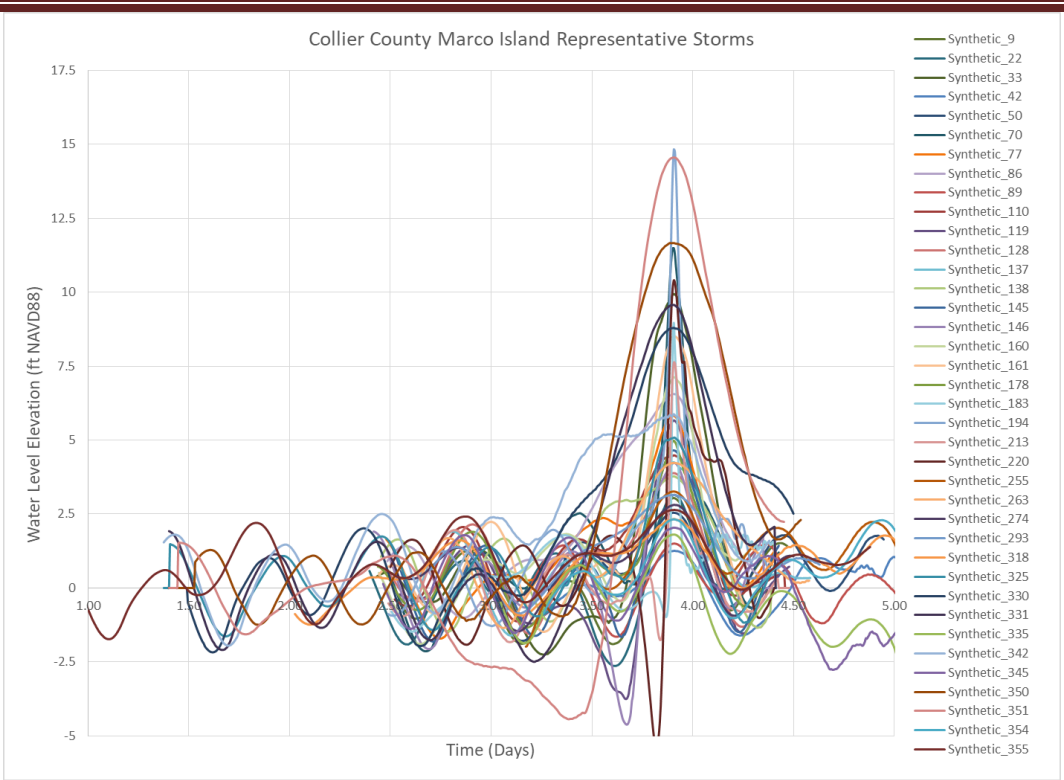


Figure 5.21 Collier County Marco Island Representative Tropical Storm Hydrographs

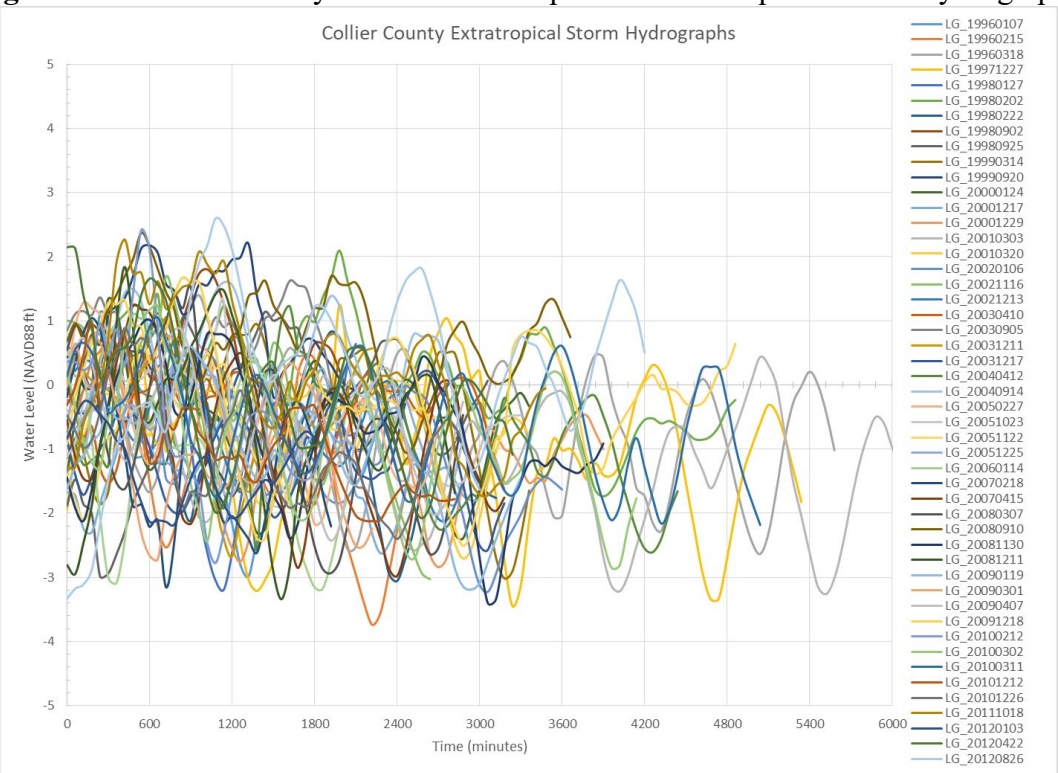


Figure 5.22 Collier County North County Extra Tropical Storm Hydrographs

Storm Seasons

In addition to the plausible storm dataset, the seasonality of the storms must also be specified. The desired storm seasons are based on the assumption that each plausible storm takes place within the season in which the original historical storm occurred. Probability is defined for each season through the Probability Parameter. The Probability Parameter is determined for each season and storm type by dividing the number of storms by the total number of years in the storm record. The Extra-Tropical seasonal storm probabilities are calculated from the historical storms derived from the WIS data covering 1980 to 2012, while the tropical storm seasons were calculated from historical storm tracks within an approximate 65 km radius of the project area. A total of 42 tropical storms from 1861 to 2017 are shown in Figure 5.23 from NOAA's Historical Hurricane Tracks Website. Table 5.7 displays the storm seasons for Collier County.

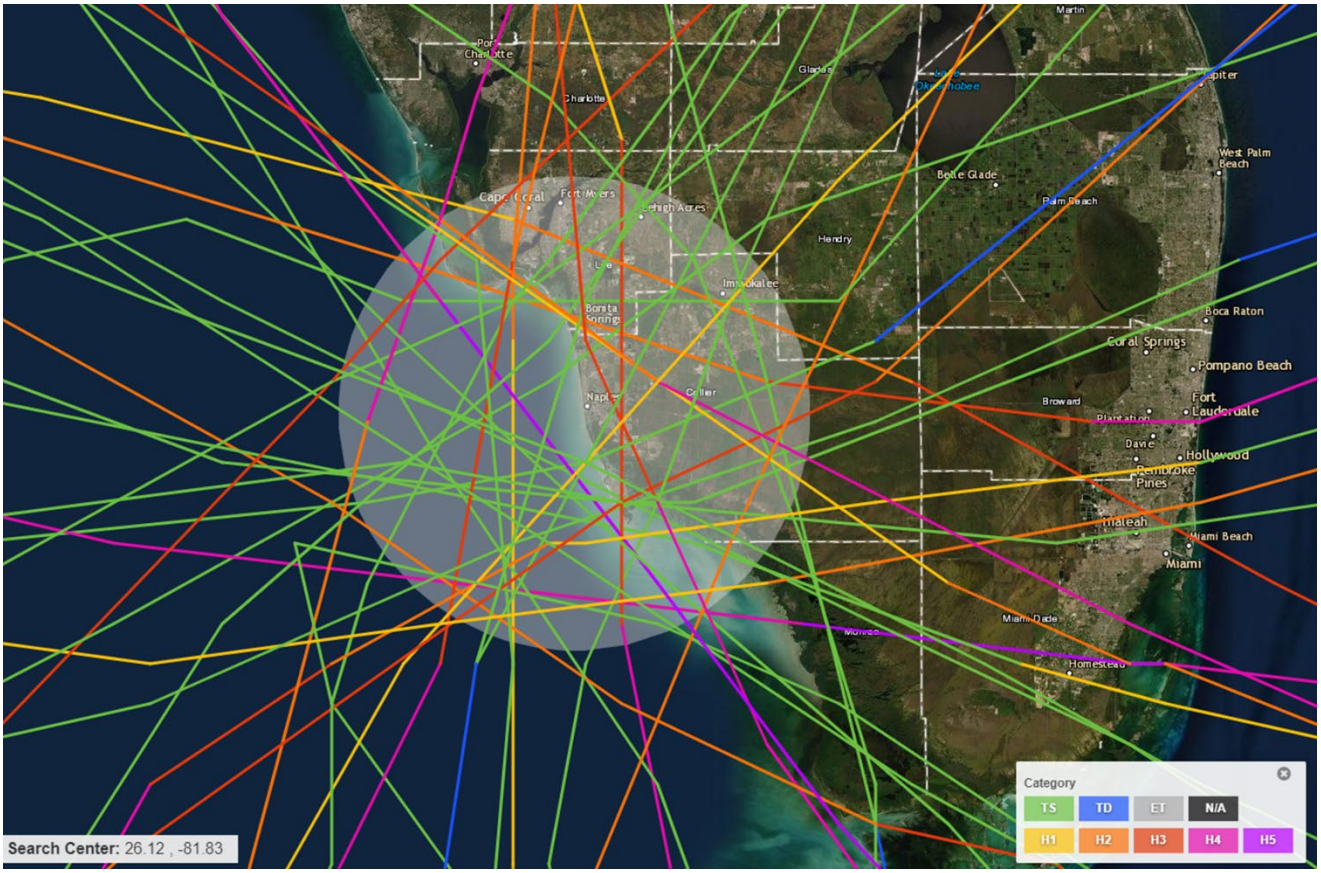


Figure 5.23 Historical Hurricane and Tropical Storm tracks near Collier County (**https://coast.noaa.gov/hurricanes/**)

Table 5.7 Collier County Beach-*fx* Storm Seasons

Storm Season	Start Date	End Date	Extra-Tropical		Tropical	
			Number	Probability	Number	Probability
Extratrop Winter/Spring	Dec 1	Apr 30	37	2.18	0	0.00
Tropical Early Summer	May 1	Jul 31	0	0.00	7	0.04
Tropical Peak	Aug 1	Sep 30	7	0.41	19	0.12

Extratrop/Tropical	Oct 1	Nov 30	5	0.29	16	0.10

The combination of the plausible storm dataset and the specified storm season allows the Beach-*fx* model to randomly select from storms of the type that fall within the season currently being processed. For each storm selected, a random time within the season is chosen and assigned as the storm date. The timing of the entire sequence of storms is governed by a pre-specified minimum storm arrival time. A minimum arrival time of 7 days was specified for Collier County. Based on this interval the model attempts to place subsequent storm events outside of a 14 day window surrounding the date of the previous storm (i.e. a minimum of 7 days prior to the storm event and a minimum of 7 days following the storm event). The model does allow the user to set different minimum arrival times for extra-tropical and tropical storms. Due to the probabilistic nature of the model the minimum arrival time may be overridden as warranted during the course of the life cycle analysis.

5.4.1.4. COASTAL MORPHOLOGY

The Beach-fx model estimates changes in coastal morphology through four primary mechanisms:

- Shoreline storm response
- Applied shoreline change
- Project-induced shoreline change
- Post-storm berm recovery

Combined, these mechanisms allow for the prediction of shoreline morphology for both with and without project conditions.

Shoreline Storm Response

Shoreline storm response is determined by applying the plausible storm set that drives the Beach-fx model to simplified beach profiles that represent the shoreline features of the project site. For this study, application of the storm set to the idealized profiles was accomplished with the SBEACH coastal processes response model (Larson and Kraus, 1989). SBEACH is a numerical model which simulates storm-induced beach change based on storm conditions, initial profiles, and shoreline characteristics such as beach slope and grain size. Output consists of post-storm beach profiles, maximum wave height and wave period information, and total water elevation including wave setup. Pre- and post-storm profiles, wave data, and water levels can be extracted from SBEACH and imported into the Beach-fx Shore Response Database (SRD). The SRD is a relational database used by the Beach-fx model to pre-store results of SBEACH simulations of all plausible storms impacting a pre-defined range of anticipated beach profile configurations

Idealized Representative Profiles

In order to develop the idealized SBEACH profiles from which the SRD was derived, it was necessary to first develop representative profiles for the project shoreline. The number of representative profiles developed for any given project depends on the natural variability of shoreline itself. Typically, historical profiles at each FDEP R-monument would be compared over time, aligned, and then averaged into a composite profile representative of the shoreline shape at that given R-monument location. Composite profiles would then be compared and separated into groupings according to the similarity between the following seven dimensions:

- Upland elevation
- Dune slope
- Dune height
- Dune width
- Berm height
- Berm width
- Foreshore slope

In order to determine the condition of the project shoreline at the model start year, historical pre-project surveys were studied. The most recent survey of most the beach areas (North County and Marco Island) is the September/October 2017 Post Hurricane Irma Monitoring survey, except from R-9 to R-22 which were surveyed as part of the 2018 Wiggins Pass Dredging Post Construction survey where dredged material are not placed in the near shore. However, as these surveys represent the beach in post construction and post storm conditions, it was decided to use the January/February 2017 monitoring surveys as the primary basis for evaluating pre nourishment beach conditions. FDEP surveys were used to fill in areas outside of the typical monitoring areas.

To develop representative profiles, beach profiles were grouped, primarily according to similar existing dune heights and widths. These beach profiles were aligned at the existing seaward toe of dune and an average profile was calculated. An example of an average profile developed for Vanderbilt Beach is shown in Figure 5.24.

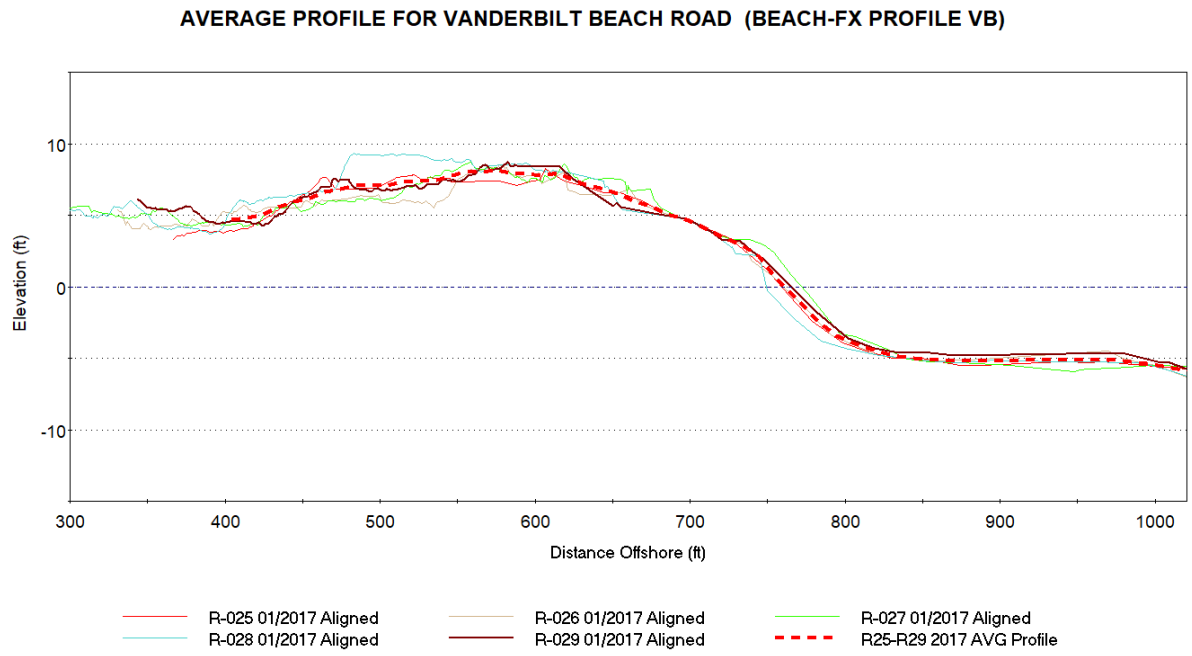


Figure 5.24 Average Profile for Vanderbilt Beach Road (Beach-*fx* Profile: VB)

When averaging profiles, one can make too many general assumptions, therefore aligned average survey profiles were also compared to pre and post construction conditions of past nourishment projects as shown in past construction drawings provided by the County. Additionally, average survey profile dune widths were also compared to analysis of survey contours and to the location of vegetation/structures on overlaid aerials (so as to ensure that the existing dune modeled was not so large as to encompass a structure) and adjusted as needed using engineering judgment for the best fit dimensions for each representative profile. All of these factors contributed to the development of the simplified representative beach profiles for Beach-*fx*. An example of how an average profile was further refined is shown in Figure 5.25 for Vanderbilt beach. While the average profiles suggests a very large, low dune, visual inspection of the aerials suggest most of this "dune" is actually where the structures are located. The dune dimensions were then adjusted so that the dune in beach-*fx* would only be located seaward of existing structures. The height of the idealized dune was also verified by assessing the highest dune elevations in front of structures at each R-monument by overlaying 2017 survey contours on an aerial and computing an average. Figure 5.25 is a good example of how the simplified profiles required in Beach-*fx* lead to major assumptions for existing conditions.

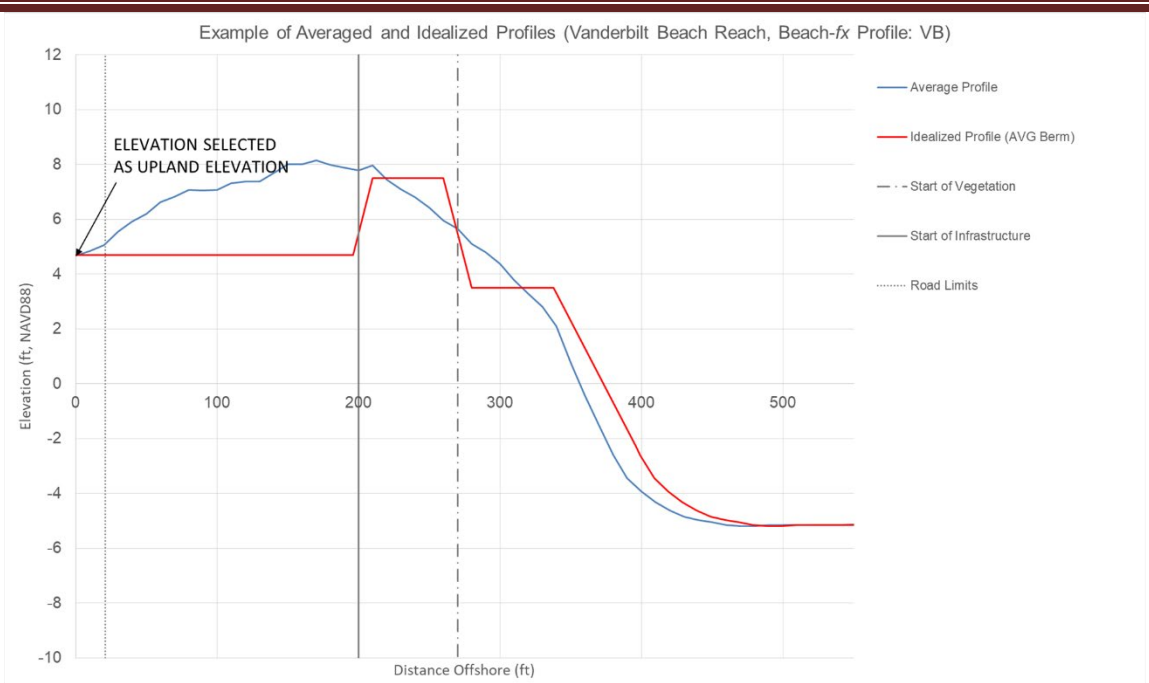


Figure 5.25 Example of Averaged and Idealized Profiles (Vanderbilt Beach Reach, Beach-*fx* Profile: VB)

Though the period of economic analysis extends from 2030 to 2079, the shoreline position of the Beach-*fx* is assumed to be the shoreline of the January/February 2017 monitoring survey i.e. in terms of model inputs 2017 would be the start year for Beach-*fx* with a base year set at 2030 and the model allowed to run for 63 years until the end of 2079. The decision to start the model at the year of the latest survey was made after feedback was provided by the model developer on intent of the start and base years in Beach-*fx*.

Using the representative profiles, idealized profiles representing the major dimensions of the beach areas profiles are defined: berm height, berm width, dune height, dune width, dune slopes, and foreshore slope. 10 representative profiles were developed for the North County beach areas (Table 5.8) and 4 representative profiles (Table 5.9) were developed for the Marco Island Beach, with the focus spent on grouping areas by similar existing dune widths and heights. Model reaches are then assigned to a profile in Beach-*fx*. Simplifying the beaches in the manner increases computational efficiency, but also presents a risk in reducing the level of detail along the beach reaches. Ideally, each R-monument would more or less have its own reach and associated representative profile, but with 89 R-monuments in the North County and 16 R-monuments on Marco Island, such detail is not feasible for this study.

It should be noted that several inputs cannot be changed between idealized profile and future planned nourishment templates including Upland Elevation, Dune Slopes, Berm Elevation, and Fore Slope, therefore some inputs were set based on what the planned nourishment templates would require, such as the 3.5 ft average construction berm based on the typical stepped “turtle

friendly berm" constructed in Collier County and 0.2 dune slopes.

Table 5.8 and Table 5.9 provide dimensions for each of the idealized Beach-*fx* profiles at the start of each Beach-*fx* simulation.

Table 5.8 Dimensions of Idealized Representative Profiles – North County Beaches

Profile	Model Reach	Upland Elevation (ft-NAVD88)	Dune Height (ft-NAVD88)	Dune Width (ft)	Dune Slopes	Berm Elevation (ft-NAVD88)	Berm Width (ft)	Fore Slope
BB1	BB1	4.7	7.6	10	0.2	3.5	30	0.13
BB2	BB2	6.5	8	95	0.2	3.5	75	0.06
WP	WP	3.6	7.2	60	0.2	3.5	60	0.11
VB	VB1, VB2	4.7	7.5	50	0.2	3.5	58	0.1
PB	PB1, PB2	3.7	6.3	60	0.2	3.5	50	0.09
CP	CP	4.3	7	15	0.2	3.5	55	0.08
PS1	PS1	4.6	11.5	10	0.2	4.5	45	0.08
PS2	PS2a, PS2b	4.7	6.5	55	0.2	3.5	50	0.09
NBGP	NB1, NB2, NB3, NB4, NB5, NB6, NB7, GP1, GP2	5	6.5	55	0.2	3.5	30	0.08
GP	GP3, GP4	4.7	6	5	0.2	3.5	55	0.13

Table 5.9 Dimensions of Idealized Representative Profiles – Marco Island Beaches

Profile	Model Reach	Upland Elevation (ft-NAVD88)	Dune Height (ft-NAVD88)	Dune Width (ft)	Dune Side Slopes	Berm Elevation (ft-NAVD88)	Berm Width (ft)	Fore Slope
MI1	MI1	4.3	8	40	0.1	5	510	0.03
MI2	MI2a, MI2b	4.7	6	60	0.1	3.5	370	0.02
MI2c	MI2c	4.7	6	5	0.1	3.5	370	0.02
MI3	MI3, MI4a	4	8.5	15	0.1	4	150	0.06
MI4	MI4b	4.5	5	10	0.2	4	30	0.04

SBEACH Methodology

SBEACH simulates beach profile changes that result from varying storm waves and water levels. These beach profile changes include the formation and movement of major morphological features such as longshore bars, troughs, and berms. SBEACH is a two-dimensional model that considers only cross-shore sediment transport. The model assumes that simulated profile changes are produced only by cross-shore processes. Longshore wave, current, and sediment transport processes are not included.

SBEACH is an empirically based numerical model, which was formulated using both field data and the results of large-scale physical model tests. Input data required by SBEACH describes the storm being simulated and the beach of interest. Basic requirements include time histories of wave height, wave period, water elevation, beach profile surveys, and median sediment grain

size.

SBEACH simulations are based on six basic assumptions:

- Waves and water levels are the major causes of sand transport and profile change
- Cross-shore sand transport takes place primarily in the surf zone
- The amount of material eroded must equal the amount deposited (conservation of mass)
- Relatively uniform sediment grain size throughout the profile,
- The shoreline is straight and longshore effects are negligible
- Linear wave theory is applicable everywhere along the profile without shallow-water wave approximations

Once applied, SBEACH allows for variable cross shore grid spacing, wave refraction, randomization of input waves conditions, and water level setup due to wind. Output data consists of a final calculated profile at the end of the simulation, maximum wave heights, maximum total water elevations plus setup, maximum water depth, volume change, and a record of various coastal processes that may occur at any time-step during the simulation (accretion, erosion, over-wash, boundary-limited run-up, and/or inundation).

SBEACH Calibration

Calibration of the SBEACH model was performed using information from Hurricane Irma (2017) (Figure 5.26). Water level information was obtained from the NOAA tide gauge at Naples Pier. This tide gauge also records wind speed and direction data, therefore the significant wave height, H_s, and dominant period, T_p, were estimated by empirical equations as shown in the following:

$$H_s = \frac{U_{max}^2}{36g}$$

$$T_p = \frac{2U_{max}}{g}$$

Where U_{max} is the maximum speed in m/s. These equations were developed and verified by Maynord et al. (2011). Figure 5.8 shows the resulting wave data by applying the measured wind speed to these equations for Hurricane Irma in 2017.

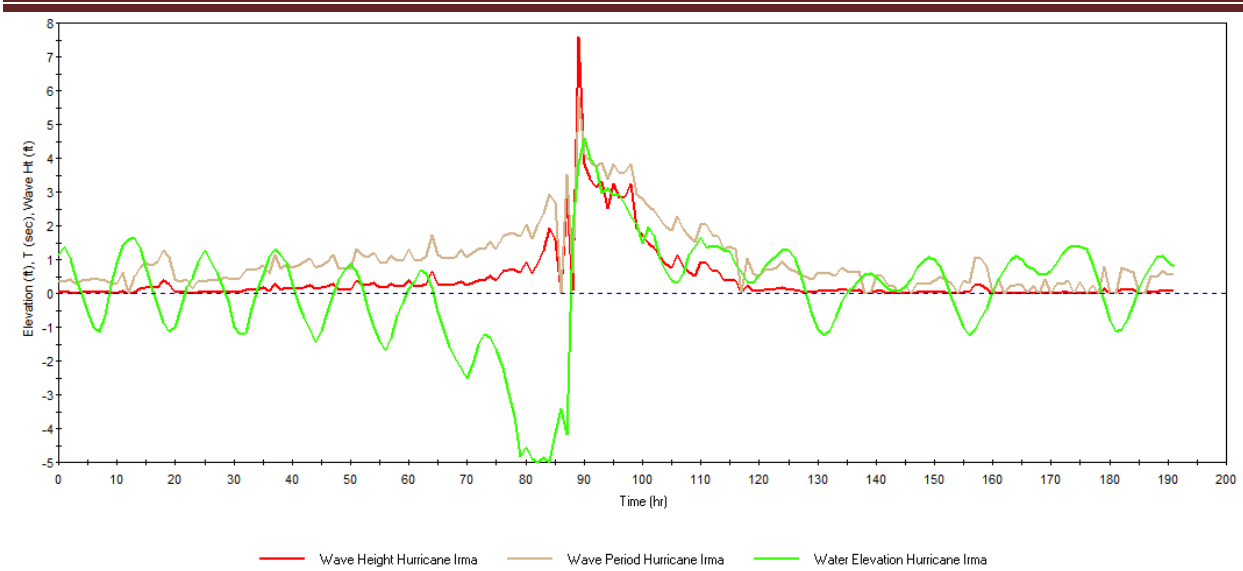


Figure 5.26 Hurricane Irma Wave and Water Level Data for SBEACH Calibration

Pre- and post-storm shoreline profiles were obtained from Collier County Monitoring Surveys. The pre storm shoreline profiles were surveyed in January/February 2017 while the post storm survey profiles were collected directly after Hurricane Irma in September 2017. The time between the surveys (6 months) is not ideal but this was the only pre and post storm dataset received for the project area.

Using the pre-storm profiles, SBEACH was then run with a range of values for an array of calibration parameters. The dry beach areas were the primary area of focus for the Sbeach calibration as Beach-*fx* ignores the submerged profile when filling the Storm Database. Table 5.10 provides the relevant beach characteristic and sediment transport calibration parameters as well as their final (best fit) calibrated values using wave height, wave period, and water level information from Hurricane Irma (2017). A few of the profiles evaluated during the calibration are shown in Figure 5.27.

Table 5.10 SBEACH Calibrated Beach Characteristic and Sediment Transport Parameters

Beach Characteristic		Sediment Transport	
Parameter	**Calibrated Value**	**Parameter**	**Calibrated Value**
Landward Surf Zone Depth	1.0 ft	Transport Rate Coefficient	1.75e-06 (m^4/N)
Effective Grain Size (See Section 2.4)	0.33 mm (North County) 0.26 (Marco Island)	Overwash Transport Parameter	0.0
		Coefficient for Slope-Dependent Term	0.002
Maximum Slope Prior to Avalanching	15	Transport Rate Decay Coefficient Multiplier	0.5
		Water Temperature	28degC

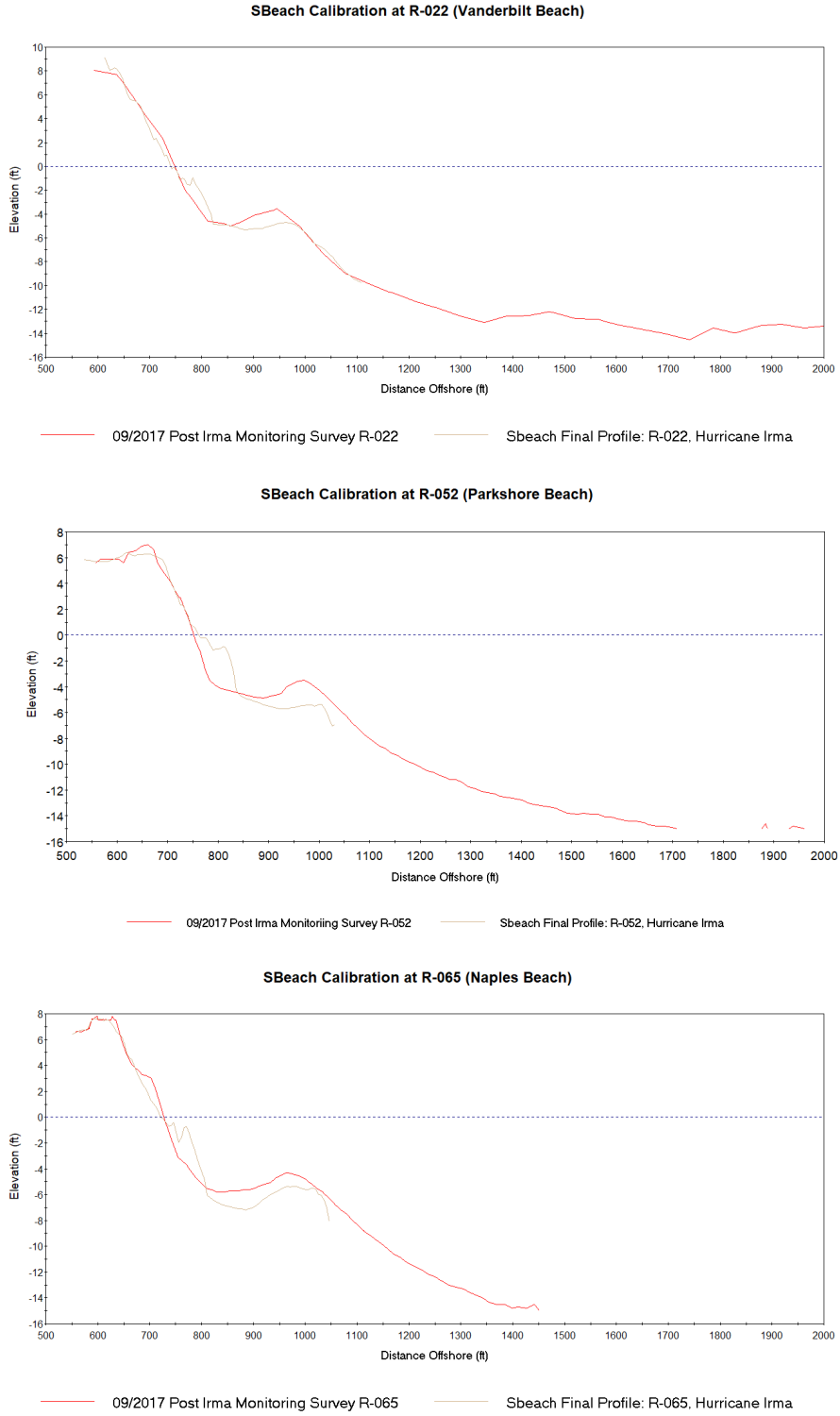


Figure 5.27 Example Sbeach Calibration Profiles

SBEACH Simulations

Calibrated Collier County North County Beaches and Marco Island SBEACH simulations were run for an array of future without project and with project (projected maximum potential dune

and berm dimensions) idealized profiles in combination with each of the tropical and extra-tropical storms in the plausible storm database. This resulted in individual storm response profiles. From these profiles, changes in the key profile dimensions were extracted and stored in the combined County North County Beaches and Marco Island Beach-*fx* SRD.

Applied Shoreline Change

The applied shoreline change rate (in feet per year) is a Beach-*fx* morphology parameter specified at each of the model reaches. It is a calibrated parameter that, combined with the storm-induced change generated internally by the Beach-*fx* model, returns the historical shoreline change rate for that location. Calibration is essential to insure that the morphology behavior is appropriate and representative of the study area.

The target shoreline change rate is an erosion or accretion rate equivalent to the historical background shoreline change rates for the project area. Changes in mean high water (MHW) position typically provide a historical view of the behavior of the shoreline. In Florida each profiles are gathered by the FDEP, local sponsors, and USACE. FDEP MHW shoreline change information is available from the 1885 – 2017 for most of the R-monuments along the North County and Marco Island. Historical shoreline erosion rates are difficult to calculate since several North County (Vanderbilt, Park Shore, Naples) and South Marco Island beach areas have been influenced by beach nourishment activities since 1996 and 1991 respectively (see Table 2.1 and Table 2.7). Additionally, the County's inlet management programs with the near shore placement of maintenance dredged material have also likely influenced background erosion rates in North County areas north and south of the inlets.

To calculate the background erosion in Beach-*fx*, the FDEP Mean High Water (MHW) change rates (ft) from 1988/1984 to present day for each R-monument along the North County Beaches and Marco Island were analyzed for a ft/yr erosion rate for that time period. The more recent FDEP MHW change rates were evaluated as it was assumed that these rates would include erosional/accretional influences of rock structures that have been constructed over the past decades. R-monument ft/yr erosion rates along the reaches were then averaged for each reach section. FDEP MHW survey dates were also compared to historical nourishment activities. For most of the nourishment activities, it was clearly seen from the dates and the change in MHW that a nourishment occurred in the FDEP change rates. Therefore, for reaches where nourishment has occurred, background erosion rates were calculated by excluding these events from the MHW change rates.

During Beach-*fx* calibration, applied erosion rates were adjusted for each model reach and the Beach-*fx* model was run for repeatedly for 300 iterations over the 50-year project life cycle. Calibration is achieved when the rate of shoreline change, averaged over hundreds of life cycle simulations, is equal to the background (target) shoreline change rate. Marco Island provided an additional challenge in that the northern reaches (R-135 – R-143), have seen high historical rates of accretion. Beach-*fx* is limited in how it can deal with high accretion rates. As these beaches also have large berms already, the decision was made to run the model with these reaches

calibrated to a change rate of 0 ft/yr for initial runs as it was anticipated these areas in their existing state would have little to no damages. If large damages were seen in these areas for the FWOP run, the model calibration would then be calibrated for an accreting beach. Table 5.11 provides the historical background erosion rates and the calibrated Beach-*fx* applied erosion rates for the North County and Marco Island Beach Reaches.

Table 5.11 Historical Background Charge Rates and Calibrated Beach-*fx* Applied Erosion Rates

North County Beaches			Marco Island Beaches		
Model Reaches	**Historical Background Change Rate (ft/year)**	**Calibrated Beach-*fx* Applied Erosion Rates (ft/year)**	**Model Reaches**	**Historical Background Change Rate (ft/year)**	**Calibrated Beach-fx Applied Erosion Rates (ft/year)**
BB1	-0.83	3.303	MI1	25.66	0.047*
BB2	-3.11	-2.608	MI2a	14.83	0.089*
WP	-4.43	0.069	MI2c	7.15	-0.01*
VB1	-1.48	2.78	MI2b	7.15	0.089*
VB2	-4.40	-0.559	MI3	-7.3	-6.793
PB1	-1.76	1.419	MI4a	-7.3	-6.793
PB2	-0.79	2.387	MI4b	-10.6	-10.883
CP	-2.63	-0.519			
PS1	-1.45	0.46			
PS2a	-2.58	0.81			
PS2b	2.60	5.959			
NB1	-7.33	-6.542			
NB2	-2.90	-1.66			
NB3	-4.78	-3.761			
NB4	-1.42	0.208			
NB5	-4.37	-3.341			
NB6	-4.01	-2.916			
NB7	-1.44	0.184			
GP1	-0.61	1.09			
GP2	-0.61	1.09			
GP4	-0.47	3.701			
GP3	-0.47	3.701			

Note: *Calibrated for 0 ft/yr change rate as these areas accrete

Post Storm Berm Recovery

Post storm recovery of eroded berm width after passage of a major storm is a recognized process. Although present coastal engineering practice has not yet developed a predictive method for estimating this process, it is an important element of post-storm beach morphology. Within Beach-*fx*, post-storm recovery of the berm is represented in a procedure in which the user specifies the percentage of the estimated berm width loss during the storm that will be recovered over a given recovery interval. It is important to note that the percentage itself is not a "stand alone" parameter that is simply applied during the post storm morphology computations. The percentage of berm recovery is estimated prior to model calibration and becomes a tunable calibration parameter to ensure model convergence (when the model reproduces the target

erosion rates as discussed in the previous section).

Based on recommendations by the model developer regarding Florida shorelines, review of available historical FDEP profiles that would qualify as pre- and post- storm, and successful model calibration a recovery percentage of 90% over a recovery interval of 21 days was determined to be appropriate for Collier County.

5.4.1.5. ECONOMIC EVALUATION

The Beach-*fx* model analyzes the economics of shore protection projects based on the probabilistic nature of storm associated damages to structures in the project area. Damages are treated as a function of structure location and construction, the intensity and timing of the storms, and the degree of protection that is provided by the natural or constructed beach. Within the model, damages are attributed to three mechanisms:

- Erosion (through structural failure or undermining of the foundation)
- Flooding (through structure inundation levels)
- Waves (though the force of impact)

Although wind may also cause shoreline damage, shore protection projects are not designed to mitigate for impacts due to wind. Therefore, the Beach-*fx* model does not include this mechanism.

Damages are calculated for each model reach, lot, and damage element following each storm that occurs during the model run. Erosion, water level, and maximum wave height profiles are determined for each individual storm from the lookup values in the previously stored SRD. These values are then used to calculate the damage driving parameters (erosion depth, inundation level, and wave height) for each damage element.

The relationship between the value of the damage driving parameter and the percent damage incurred from it is defined in a user-specified "damage function". Two damage functions are specified for each damage element, one to address the structure and the other to address its contents. Damages due to erosion, inundation, and wave attack are determined from the damage functions and then used to calculate a combined damage impact that reduces the value of the damage element. The total of all FWOP damages is the economic loss that can be mitigated by the shore protection project.

A thorough discussion of the economic methodology and processes of Beach-*fx* can be found in the Economics Appendix.

5.4.1.6. MANAGEMENT MEASURES

Shoreline management measures that are provided for in the Beach-*fx* model are emergency nourishment and planned nourishment.

Emergency Nourishment

Emergency nourishments are generally limited beach fill projects conducted by local governments in response to storm damage. Collier County does have a history of what might be referred to as emergency nourishment, several of the truck haul nourishment projects listed in Table 2.1 were in response to storm related erosion. However, Emergency Nourishment is limited in Beach-*fx* in that the model is currently limited to a specification of a nourishment fill density (cubic yard/feet) that acts to increase dune width (at the current dune elevation) and the expense of berm width. The berm is only filled if the beach morphology is in a scarping condition, in which case the fill material is first used to restore the berm for the deficit volume represented by the scarping condition. Currently, Emergency Nourishment is not able to be triggered by a specific reduction in dune height or berm width (USACE, 2009). As a reduction in berm width the primary trigger for existing nourishments. Therefore, no emergency management measure was included in the Beach-*fx* analysis as it was decided they would not be the most accurate method for modeling existing nourishment conditions.

Planned Nourishment

Planned nourishments are handled by the Beach-*fx* model as periodic events based on nourishment templates, triggers, and nourishment cycles. Nourishment templates are specified at the model reach level and include all relevant information such as order of fill, dimensions, placement rates, unit costs, and borrow-to-placement ratios. Planned nourishments occur at a user defined nourishment interval when user defined nourishment triggers (expressed as a percent of specified nourishment template) are exceeded and a mobilization threshold volume is met. It should be noted that Beach-*fx* does not differentiate between a construction template and a design template. At a pre-set interval, all model reaches which have been identified for planned nourishment are examined. In reaches where one of the nourishment threshold triggers is exceeded, the required volume to restore the design template is computed. If the summation of individual model reach level volumes exceeds the mobilization threshold volume established by the user, then nourishment is triggered and all model reaches identified for planned nourishment are restored to the nourishment template. Planned nourishment were the primary management measure used in Beach-*fx* analysis for Collier County.

Nourishment Templates Requirements

Below is a list of the primary parameters required for Beach-*fx* nourishment measures:

- **Production Rate (cubic yards/day)**
  - The rate at which fill volume is placed on the beach to construct the Planned Nourishment project expressed in units of cubic yards per day.
- **Borrow To Placement Ratio**
  - The estimated volume of borrow material required to produce a unit volume of stable fill material on the project beach. This ratio is often referred to as the overfill ratio and accounts for volumetric losses due the sorting and winnowing of fines contained in the fill material.

- **Dune Height (feet) (nourishment trigger)**
  - Fractional amount of template dune height that denotes requirement for renourishment.
- **Dune Width (feet) (nourishment trigger)**
  - Fractional amount of template dune width that denotes requirement for renourishment.
- **Berm Width (feet) (nourishment trigger)**
  - Fractional amount of template berm width that denotes requirement for renourishment.
- **Dune Height (feet) (template)**
  - The post-construction dune height.
- **Dune Width (feet) (template)**
  - The post-construction dune width.
- **Berm Width (feet) (template)**
  - The post-construction berm width.

5.4.1.7. BEACH-FX PROJECT SCENARIOS

Beach-*fx* was used to run Future With Project Scenarios and Future With Project Scenarios. All the project scenarios were run over a 50 year project life with an intermediate rate of SLR. *The following sections are to be updated with results of optimized Beach-*fx* runs

Future Without Project (FWOP) Scenarios

The County has conducted existing nourishment activities for the past two decades (see Table 2.1 and Table 2.7), however, it will be assumed that in the future, the County will be less able over time to maintain the current condition of the beach, due to future economic impacts of SLR, the increased severity of future storms hitting the area requiring funds to be diverted, dwindling sand sources, etc. The reduction in possible future nourishments by the County are difficult to quantify and there is no guarantee that the County will continue to place sand on their beaches from here on out. Therefore, for the FWOP scenario is assumed that no nourishment shall occur in the future. See the Main Report and the Economics Appendix for more information justifying the FWOP scenario decision.

Future With Project (FWP) Planned Nourishment Scenarios

19 FWP scenarios were run for the North County beaches. These scenarios covered potential nourishment configurations at with varying dune heights and berm widths. The dune width was kept constant at 20 ft for each scenario, even those FWP scenarios set to the existing dune height. The justification behind this was to test for whether or not wider berm scenarios with smaller dunes, even than existing, would provide for more benefits than a taller dune scenarios with narrower berms. Figure 5.28 is provided to demonstrate how the FWP templates compare to an initial idealized profile using Vanderbilt Beach as an example. Beach-*fx* keeps the landward toe of the dune constant between all scenarios, which is similar to how a new beach should be designed at Collier County as there is not much space to move a dune landward. Therefore

Figure 5.28 demonstrates that the FWP scenarios with a higher dune will result in a wider overall dune footprint and the berm will extend further seaward. This potential FWP footprint is important to consider even in early stages of modeling and design as there is hard bottom offshore many Collier County North County areas (See 3.9 NEARSHORE HARDBOTTOM) and wider beach systems (dunes + berm footprints) may result in a need for reef habitat mitigation (See the Environmental Appendix for more information).

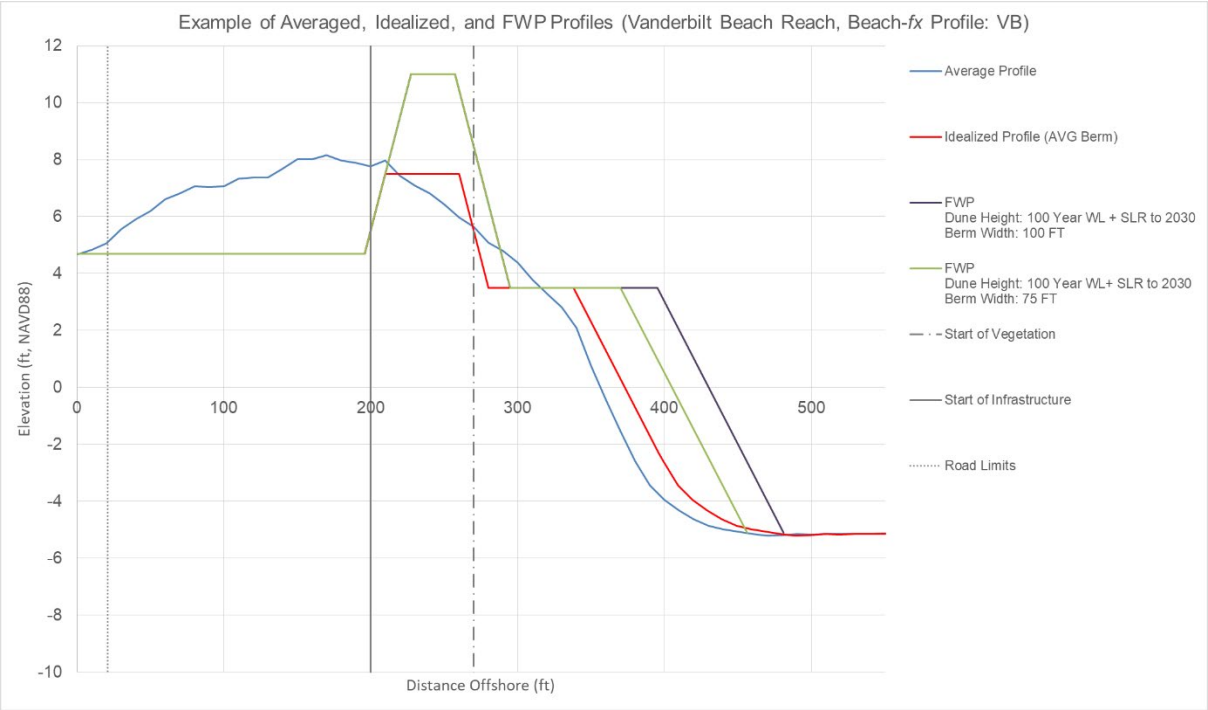


Figure 5.28 Example of Averaged and Idealized Profiles vs FWP Profiles (Vanderbilt Beach Reach, Beach-fx Profile: VB)

The North County FWP scenarios are described in Table 5.12 and the nourishment inputs are detailed in Table 5.13 for each beach reach. For each beach reach four dune elevations were considered: the existing dune height, 10 ft elevation, 12 ft elevation, and 14 ft elevation. The berm widths considered at each of the dune heights were 75, 100, 125, and 150 ft, widths similar to existing nourishment templates and extending approximately 50 – 75 ft beyond existing nourishment widths and the County wanted to evaluate the potential to increase their existing nourishment templates. For some cases a 50 ft berm width was also considered.

The template triggers were all set to 0.5 (50%) for each scenario, and the nourishments were set to be triggered every 7 years, a typical beach nourishment interval in Florida. A test was also performed to explore the potential for setting a Mobilization Trigger of 1,000,000 CY and the nourishment interval to 1 year (essentially the model will check every year for nourishment but will only trigger if the estimated volume exceeds the nourishment trigger and at least one template variable is triggered) to see how often nourishments would be triggered. The test on several scenarios resulted in nourishments every 3-4 years and a significant increase in overall volume. Ultimately is was decided to continue the modeling of the scenarios with the 7 year

interval.

Table 5.12 North County FWP Beach-*fx* Scenarios Descriptions

Number	Scenario Name	Scenario Description
1	DW20_H_E_B75	Dune heights set to existing dune height elevation, dune width set to 20 ft, berm width set to 75 ft
2	DW20_H_E_B100	Dune heights set to existing dune height elevation, dune width set to 20 ft, berm width set to 100 ft
3	DW20_H_E_B125	Dune heights set to existing dune height elevation, dune width set to 20 ft, berm width set to 125 ft
4	DW20_H_E_B150	Dune heights set to existing dune height elevation, dune width set to 20 ft, berm width set to 150 ft
5	DW20_10FT_B50	Dune heights set to 10FT elevation, dune width set to 20 ft, berm width set to 50 ft
6	DW20_10FT_B75	Dune heights set to 10FT elevation, dune width set to 20 ft, berm width set to 75 ft
7	DW20_10FT_B100	Dune heights set to 10FT elevation, dune width set to 20 ft, berm width set to 100 ft
8	DW20_10FT_B125	Dune heights set to 10FT elevation, dune width set to 20 ft, berm width set to 125 ft
9	DW20_10FT_B150	Dune heights set to 10FT elevation, dune width set to 20 ft, berm width set to 150 ft
10	DW20_12FT_B150	Dune heights set to 12 FT elevation for all reaches, dune width set to 20 ft, berm width set to 150ft
11	DW20_12FT_B125	Dune heights set to 12 FT elevation for all reaches, dune width set to 20 ft, berm width set to 125ft
12	DW20_12FT_B100	Dune heights set to 12 FT elevation for all reaches, dune width set to 20 ft, berm width set to 100ft
13	DW20_12FT_B75	Dune heights set to 12 FT elevation for all reaches, dune width set to 20 ft, berm width set to 75ft
14	DW20_12FT_B50	Dune heights set to 12 FT elevation for all reaches, dune width set to 20 ft, berm width set to 50ft
15	DW20_14FT_B150	Dune heights set to 14 FT elevation for all reaches, dune width set to 20 ft, berm width set to 150ft
16	DW20_14FT_B125	Dune heights set to 14 FT elevation for all reaches, dune width set to 20 ft, berm width set to 125ft
17	DW20_14FT_B100	Dune heights set to 14 FT elevation for all reaches, dune width set to 20 ft, berm width set to 100ft
18	DW20_14FT_B75	Dune heights set to 14 FT elevation for all reaches, dune width set to 20 ft, berm width set to 75ft
19	DW20_14FT_B50	Dune heights set to 14 FT elevation for all reaches, dune width set to 20 ft, berm width set to 50ft

Table 5.13 North County FWP Beach-*fx* reach plan nourishment inputs

Reach	Production Rate (CY/day)	BorrowToPlacementRatio	Nourishment Triggers (%			Berm Width (FT)	Dune Width (FT)	Dune Heights			
			Berm Width	Dune Width	Dune Height			Existing	10 FT	12 FT	14 FT
BB1	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	7.6	10	12	14
BB2	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	8	10	12	14
WP	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	7.2	10	12	14
VB1	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	7.5	10	12	14
VB2	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	7.5	10	12	14
PB1	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.3	10	12	14
PB2	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.3	10	12	14
CP	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	7	10	12	14
PS1	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	11.7	10	12	14
PS2a	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.5	10	12	14
PS2b	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.5	10	12	14
NB1	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.7	10	12	14
NB2	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.7	10	12	14
NB3	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.7	10	12	14
NB4	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.7	10	12	14
NB5	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.7	10	12	14
NB6	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.7	10	12	14
NB7	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.7	10	12	14
GP1	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.7	10	12	14
GP2	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6.7	10	12	14
GP4	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6	10	12	14
GP3	7400	1.2	0.5	0.5	0.5	50*,75,100,125,150	20	6	10	12	14

*50 FT Berms were only evaluated with 10 FT, 12 FT and 14 FT Dune Heights.

The quantities for a few FWP scenarios are presented below. These are the FWP scenarios that provided the highest cost benefit ratios (see the Economics Appendix) and were optimized further to select a TSP FWP beach scenario. Note that the table highlights the beaches in Planning Areas 1 and 3 as initial economics analysis shows these planning areas have the greatest potential to be included in the TSP.

Table 5.14 Estimated FWP Nourishment Quantities

			FWP_10FT_75FT			FWP_12FT_75FT			FWP_14FT_75FT		
				Renourishment			Renourishment			Renourishment	
			Initial Nourishment [CY]	Avg Renourishment [CY]	Total of Renourishment [CY]	Initial Nourishment [CY]	Avg Renourishment [CY]	Total of Renourishment [CY]	Initial Nourishment [CY]	Avg Renourishment [CY]	Total of Renourishment [CY]
Planning Area 1	Approx Length [Miles]	AVG NUM EVENTS	1	7		1	7		1	7	
	1.54	Barefoot Beach (R1 - R9)	572,600	195,014		749,800	205,714		939,800	224,786	
	1.42	Barefoot Beach Preserve (R9-R16)	81,400	262,371		41,400	274,300		199,800	290,500	
	1.16	Wiggins Pass (R17 - R22)	208,500	382,771		343,300	398,271		490,900	421,157	
	1.31	Vanderbilt (R22 - R29)	227,600	259,657		378,100	272,729		544,000	285,243	
	5.42	Total	1,090,100	1,099,814	7,698,700	1,512,600	1,151,014	8,057,100	2,174,500	1,221,686	8,551,800
Planning Area 2		AVG NUM EVENTS	1	7		1	7		1	7	
	2.30	Pelican Bay (R29 - R41)	396,400	450,329		662,800	471,657		956,900	495,643	
	1.03	Clam Pass (R42 - R46)	362,200	219,457		484,800	225,786		618,800	236,657	
	3.33	Total	758,600	669,786	4,688,500	1,147,600	697,443	4,882,100	1,575,700	732,300	5,126,100
Planning Area 3		AVG NUM EVENTS	1	7		1	7		1	7	
	2.15	Park Shore (R46 - R57)	343,600	284,629		593,000	300,457		859,800	318,557	
	2.01	Naples (R 58A -R 68)	728,500	525,157		978,500	546,071		1,248,800	577,886	
	4.16	Total	1,072,100	809,786	5,668,500	1,571,500	846,529	5,925,700	2,108,600	896,443	6,275,100
Planning Area 4		AVG NUM EVENTS	1	7		1	7		1	7	
	1.79	Naples (R68 - R 79)	616,500	269,657		837,500	282,014		1,083,000	298,743	
	1.83	Gordon Pass (R79-R89)	484,800	312,043		696,000	324,486		931,700	331,671	
	3.62	Total	1,101,300	581,700	4,071,900	1,533,500	606,500	4,245,500	2,014,700	630,414	4,412,900
		Total CY	4,022,100	3,161,086	22,127,600	5,765,200	3,301,486	23,110,400	7,873,500	3,480,843	24,365,900
		PA 1 & 3 Total	2,162,200	1,909,600	13,367,200	3,084,100	1,997,543	13,982,800	4,283,100	2,118,129	14,826,900
		Nourishment Frequency (years)		**7**			**7**			**7**	

Nine (9) FWP scenarios were run for the Marco Island beaches. These scenarios covered potential nourishment configurations at with varying dune heights and berm widths. The dune width was kept constant at 20 ft for each scenario. The Marco Island FWP scenarios are described in Table 5.15 and the nourishment inputs are detailed in Table 5.16 for each beach reach. For each beach reach three dune elevations were considered: the existing dune height, the dune height equivalent to the FEMA SWFL Study 50 year SWL elevation (8 FT), and the dune height equivalent to the FEMA SWFL Study 100 year SWL elevation (10 FT). As Marco Island contains two reaches which show historical accretion, MI1 and MI2, only the dune heights were varied for these scenarios. Berm widths were varied for the erosional South Marco Island reaches which have been nourished in the past. MI4b was the only reach were extending the berm past existing nourishment widths was considered, as MI3 and MI4a are currently nourished to a width of 300 ft as measured from past construction documents. The template triggers were all set to 0.5 (50%) for each scenario, and the nourishments were set to be triggered every 7 years, a typical beach nourishment interval in Florida.

Table 5.15 Marco Island FWP Beach-*fx* Scenario Descriptions

Scenario Name	Scenario Description
FWP_E_Max	Dune heights set to existing dune height elevation, dune width set to 20 ft, berm width set to maximum width, excluding MI1 and MI2 reaches which do not vary width.
FWP_E_Med	Dune heights set to existing dune height elevation, dune width set to 20 ft, berm width set to median width, excluding MI1 and MI2 reaches which do not vary width.
FWP_E_Min	Dune heights set to existing dune height elevation, dune width set to 20 ft, berm width set to minimum width, excluding MI1 and MI2 reaches which do not vary width.
FWP_50YR_Max	Dune heights set to 50 YR SWL elevation, dune width set to 20 ft, berm width set to maximum width, excluding MI1 and MI2 reaches which do not vary width.
FWP_50YR_Med	Dune heights set to 50 YR SWL elevation, dune width set to 20 ft, berm width set to median width, excluding MI1 and MI2 reaches which do not vary width.
FWP_50YR_Min	Dune heights set to 50 YR SWL elevation, dune width set to 20 ft, berm width set to minimum width, excluding MI1 and MI2 reaches which do not vary width.
FWP_100YR_Max	Dune heights set to 100 YR SWL elevation, dune width set to 20 ft, berm width set to maximum width, excluding MI1 and MI2 reaches which do not vary width.
FWP_100YR_Med	Dune heights set to 100YR SWL elevation, dune width set to 20 ft, berm width set to median width, excluding MI1 and MI2 reaches which do not vary width.
FWP_100YR_Min	Dune heights set to 100 YR SWL elevation, dune width set to 20 ft, berm width set to minimum width, excluding MI1 and MI2 reaches which do not vary width.

Table 5.16 Marco Island FWP Beach-*fx* reach plan nourishment inputs

Reach	Production Rate (CY/day)	Borrow ToPlacementRatio	Nourishment Triggers (%)			Nourishment Template (ft)				
			Berm Width	Dune Width	Dune Height	Berm Width	Dune Width	Dune Height		
								Existing	50 YR SWL	100 YR SWL
MI1	2500	1.2	0.5	0.5	0.5	500	20	8	8	10
MI2a	2500	1.2	0.5	0.5	0.5	350	20	6	8	10
MI2b	2500	1.2	0.5	0.5	0.5	350	20	6	8	10
MI2c	2500	1.2	0.5	0.5	0.5	350	20	6	8	10
MI3	2500	1.2	0.67	0.5	0.5	250, 275, 300	20	8.5	8	10
MI4a	2500	1.2	0.5	0.5	0.5	250, 275, 300	20	5	8	10
MI4b	2500	1.2	0.5	0.5	0.5	100, 125, 150	20	5	8	10

None of the Marco Island Runs produced enough benefits compared to the FWOP with dredging scenario for be considered further as part of the TSP. See the Economics Appendix for more information.

5.4.1.8. RECOMMENDATION

Based on the results comparing the benefits between the FWOP and the FWP scenarios, the 12 ft NAVD88 high dune with a 75 ft wide berm provided the most benefits as a beach nourishment scenario. This is the recommended beach nourishment template for all beach areas included in the TSP. See the Economics Appendix for a discussion on the Costs and Benefits of the beach nourishment.

Note: This recommendation is subject to change after additional model runs.

5.4.2. SAND SOURCES AVAILABLE FOR BEACH NOURISHMENT

Sand sources for beach re-nourishment will typically come from either off shore dredging or local quarries and mines. Beach re-nourishment material will be expected to be similar to the following specifications:

- Maximum Shell Content: 1% retained on the No. 4 sieve
- Munsell Color Value: Moist Value (Chroma = 1) of 7 or lighter
- Median grain size: 0.33 millimeters

Note: These values may change based on possible available sources or sponsor preference.

According to the recent renourishment contract from 2016, Collier County required a mean grain size of 0.33 millimeters for placed renourishment sands. A QA/QC program will be provided throughout construction to verify the fill material placed at re-nourishment areas.

A letter was provided by the County (and attached as an Appendix) to the Norfolk District detailing their available sand sources for the next 50 years to cover their existing and estimated future nourishment requirements. These sources are summarized in paragraph 2.4.1.1 BORROW

AREA CHARACTERISTICS in this report. The borrow area that the County currently leases from BOAM (T1) likely has enough material for the initial nourishment of the recommended beach planning area and potentially enough for a renourishment as well. However, this borrow area does not have enough material to supply the renourishments throughout the 50 year project life. Additional borrow areas area mentioned in the letter as T1 Expanded and T2. Though these are known shoals in a geological feature similar to T1, information on these additional sources is sparse. What is known is discussed in the Environmental Appendix. There is risk associated with assuming the use of these additional borrow sites for future renourishments after the initial project. One of the first tasks during PED should be performing a sand source investigation to determine the suitability and availability of these assumed sources.

5.5. BACK BAY MEASURES ANALYSIS

5.5.1. FLOODWALLS

Based on the anticipated flood water levels and local geology it was determined that T-walls would be appropriate to use in the alignments as existing conditions permitted. Wall heights varied from 1 feet to 30 feet. ETL 1110-2-575 and ECB 2017-03 advises for the use of I-walls where the height of wall is to be 6 feet or less. However due to the presence of bedrock close to the ground surface, sheet pile-based I-walls would not be feasible because sheet piles cannot be driven into bedrock. Secant pile walls were considered as an alternative to I-walls for wall heights 6 feet or less. These consist of interlocking auger cast piles that are able to penetrate the local geology to provide a structurally feasible alternative. I-walls were conceptualized with an auger cast secant pile base for this project. Figure 5.29 shows a graphical image of a typical I-wall with sheet pile base. Figure 5.30 is an image of an existing I-wall in the City of Norfolk, Virginia.

For the estimating purposes of this study, it was determined that T-walls would be used for all wall heights. This decision provides conservative estimates in regards to construction and real estate impacts. Further study and design are required to determine the most effective wall type and section for wall heights 6 feet or less. T-Walls will be traditional concrete stem walls with pile supported bases. Piles will be both battered and straight and are anticipated to be 1.5 feet in diameter and approximately 20 feet in length. T-walls will be designed in accordance with EM 1110-2-2502. For a more detailed discussion on proposed floodwalls refer to the Geotechnical and Structural Sub-Appendices. Figure 5.31 shows a graphical image of a typical T-wall above ground and foundation below ground. Figure 5.32 is an image of a T-wall floodwall from the Richmond Flood Control Project, an existing U.S. Army Corps of Engineers Norfolk District Project. See the structural sub-appendix for T-wall drawings.

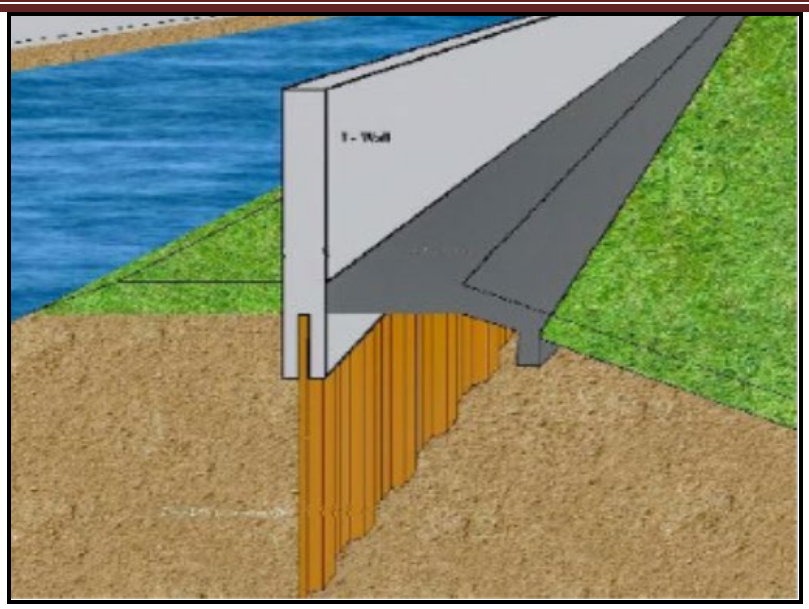


Figure 5.29 Graphical drawing of a typical sheet pile-based I-wall cross section (Image courtesy of Tulane University)

Figure 5.30 Existing I-wall located in the City of Norfolk.

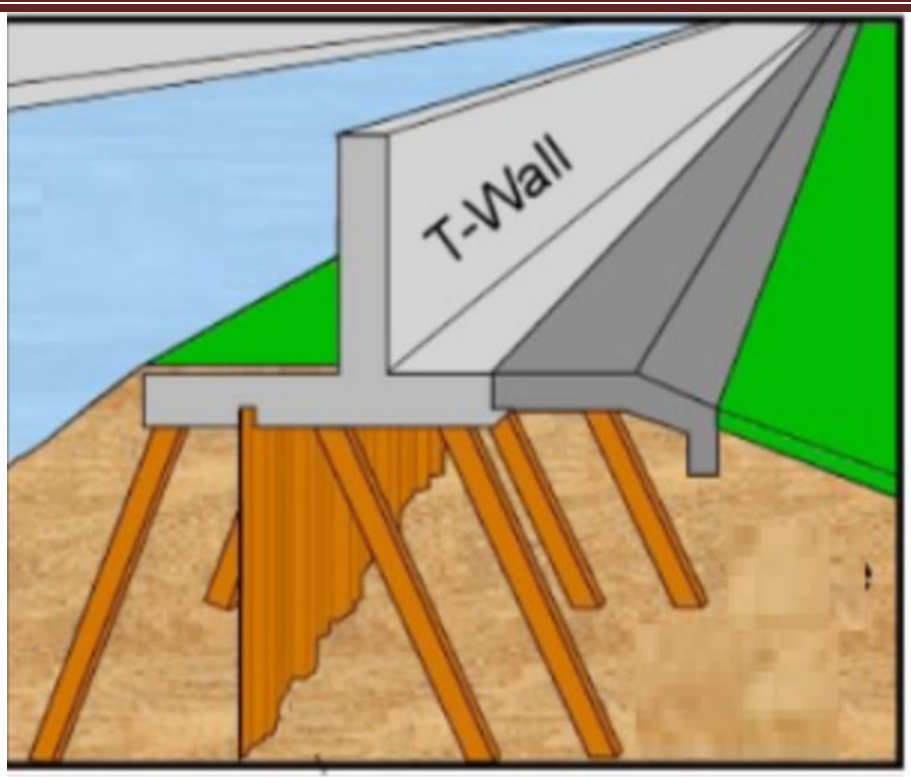


Figure 5.31 Graphical drawing of a typical T-wall cross-section (Image courtesy of Tulane University)

Figure 5.32 Existing T-wall located at the Richmond, VA Flood Control Project

5.5.2. GATE CLOSURES

The road gate closures will be steel plates on frames that will slide on rails to close for a storm event but will otherwise be stored in recessed pockets in the concrete walls to permit the

flow of traffic. There were approximately 62 locations identified, for each Top of Wall (T.O.W.) Elevation design level (10.0, 12.0, 14.0 ft. NAVD88), for potential road gate closures at the inland alignments (Table 5.17 to Table 5.25). The closures are envisioned to be similar in operation and size to those currently in use in the existing Norfolk Floodwall located in the downtown area of Norfolk, VA. The stem heights of the closures were all approximately 2 feet to 20 feet above grade. We extrapolated estimates of construction from the as-built drawings of the existing flood gates used in downtown Norfolk. Figure 5.33 and Figure 5.34 are images of a gate closure that is a part of the Norfolk Floodwall.

Table 5.17 Bonita Beach Road Alignment Proposed Gate Closures (T.O.W. 14 FT NAVD88)

PRELIMINARY STREET CLOSURES COLLIER COUNTY CSRM 10% DESIGN – Bonita Beach Road							
No.	**Location**		**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
1	Barefoot Beach Boulevard		10+15	2.0	14.0	12.0	150
2	Private Entrance		15+85	1.5	14.0	12.5	40
3	Private Entrance		33+70	1.5	14.0	12.5	60
4	Private Entrance		40+90	3.0	14.0	11	55
5	Private Entrance		48+00	5.0	14.0	9	65
6	Private Entrance		50+20	6.0	14.0	8	40
7	Private Entrance		50+80	6.0	14.0	8	30
8	Private Entrance		52+15	6.0	14.0	8	60
9	Private Entrance		56+00	7.0	14.0	7	50
10	Private Entrance		57+65	7.5	14.0	6.5	30
11	West Avenue		60+25	8.0	14.0	6	110

Table 5.18 Seagate Drive Alignment Proposed Gate Closures (T.O.W. 14 FT NAVD88)

PRELIMINARY STREET CLOSURES COLLIER COUNTY CSRM 10% DESIGN – Seagate Drive							
No.	**Location**		**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
1	Crayon Road		40+65	5.2	14.0	8.8	70

Table 5.19 Tamiami Trail Alignment Proposed Gate Closures (T.O.W. 14 FT NAVD88)

PRELIMINARY STREET CLOSURES COLLIER COUNTY CSRM 10% DESIGN – Tamiami Trail							
No.	**Location**		**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
1	4th Avenue South		3+75	6.6	14.0	7.4	50
2	5th Avenue Parkway		5+75	6.6	14.0	7.4	85
3	(2x) Private Entrance		10+65	3.6	14.0	11.4	25
4	Tamiami Trail (SB)		11+62	2.6	14.0	11.4	50

PRELIMINARY STREET CLOSURES COLLIER COUNTY CSRM 10% DESIGN – Tamiami Trail							
No.	**Location**		**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
	Tamiami Trail (NB)		12+11	2.6	14.0	11.4	70
6	10th Street South		16+50	-0.4	14.0	14.4	65
7	(2x) Private Entrance		20+15	-3.4	14.0	17.4	35
8	Private Entrance		22+40	-4.4	14.0	18.4	25
9	Goodlette Frank Road (SB)		24+95	-6.4	14.0	20.4	65
	Goodlette Frank Road (NB)		25+50	-6.4	14.0	20.4	65
11	River Point Drive		35+60	-4.4	14.0	18.4	55
12	Private Entrance		47+25	4.6	14.0	9.4	65
13	Private Entrance		51+85	4.6	14.0	9.4	80
14	Davis Boulevard (WB)		55+35	4.6	14.0	9.4	70
	Davis Boulevard (EB)		56+40	4.6	14.0	9.4	70
16	Entrance to Davis Boulevard		58+00	4.6	14.0	9.4	90
17	Private Entrance		62+65	4.6	14.0	9.4	45
18	(2x) Private Entrance		63+50	4.6	14.0	9.4	40
19	(2x) Private Entrance		65+05	4.6	14.0	9.4	35
	(2x) Private Entrance		66+30	4.6	14.0	9.4	25
21	(2x) Private Entrance		67+35	4.6	14.0	9.4	40
22	(2x) Private Entrance		69+80	4.6	14.0	9.4	35
23	Private Entrance		71+90	4.6	14.0	9.4	55
24	Commercial Drive		73+00	4.6	14.0	9.4	80
	Private Entrance		75+75	4.6	14.0	9.4	40
26	Private Entrance		79+25	4.6	14.0	9.4	25
27	(2x) Private Entrance		79+75	4.6	14.0	9.4	55
28	(2x) Private Entrance		82+90	4.6	14.0	9.4	40
29	Private Entrance		86+35	4.6	14.0	9.4	45
	Private Entrance		89+50	4.6	14.0	9.4	35
31	Private Entrance		92+35	4.6	14.0	9.4	30
32	Pelton Avenue		94+30	4.6	14.0	9.4	100
33	Private Entrance		96+00	4.6	14.0	9.4	35
34	Pineland Street		98+50	4.6	14.0	9.4	105
	(2x) Private Entrance		99+75	4.6	14.0	9.4	22
36	(2x) Private Entrance		101+05	4.6	14.0	9.4	25
37	Private Entrance		102+65	4.6	14.0	9.4	20
38	Private Entrance		103+50	4.6	14.0	9.4	40
39	Bayshore Drive		105+10	4.6	14.0	9.4	100
	Bayside Street		109+35	4.6	14.0	9.4	105
41	Private Entrance		111+10	5.1	14.0	8.9	35
42	Andrew Drive		113+40	5.1	14.0	8.9	75
43	Private Entrance		114+90	5.1	14.0	8.9	30
44	(2x) Private Entrance		116+10	5.1	14.0	8.9	20
	(2x) Private Entrance		117+15	5.1	14.0	8.9	30
46	Airport Pulling Road South		122+45	5.2	14.0	8.8	175
47	(2x) Private Entrance		127+50	5.2	14.0	8.8	30
48	Private Entrance		129+70	5.2	14.0	8.8	50
49	(2x) Espinal Boulevard		133+95	5.2	14.0	8.8	50
	Private Entrance		138+15	5.2	14.0	8.8	100

Table 5.20 Bonita Beach Road Alignment Proposed Gate Closures (T.O.W. 12 FT NAVD88)

PRELIMINARY STREET CLOSURES COLLIER COUNTY CSRM 10% DESIGN – Bonita Beach Road

No.	Location		Station	Sill Elevation (ft.)	Elevation of Top of Gate (ft.)	Height (ft.)	Width (ft.)
1	Barefoot Beach Boulevard		10+15	2.0	12.0	10.0	150
2	Private Entrance		15+85	1.5	12.0	10.5	40
3	Private Entrance		33+70	1.5	12.0	10.5	60
4	Private Entrance		40+90	3.0	12.0	9	55
5	Private Entrance		48+00	5.0	12.0	7	65
6	Private Entrance		50+20	6.0	12.0	6	40
7	Private Entrance		50+80	6.0	12.0	6	30
8	Private Entrance		52+15	6.0	12.0	6	60
9	Private Entrance		56+00	7.0	12.0	5	50
10	Private Entrance		57+65	7.5	12.0	4.5	30
11	West Avenue		60+25	8.0	12.0	4	110

Table 5.21 Seagate Drive Alignment Proposed Gate Closures (T.O.W. 12 FT NAVD88)

PRELIMINARY STREET CLOSURES COLLIER COUNTY CSRM 10% DESIGN – Seagate Drive							
No.	Location		Station	Sill Elevation (ft.)	Elevation of Top of Gate (ft.)	Height (ft.)	Width (ft.)
1	Crayon Road		40+65	5.2	12.0	6.8	70

Table 5.22 Tamiami Trail Alignment Proposed Gate Closures (T.O.W. 12 FT NAVD88)

PRELIMINARY STREET CLOSURES COLLIER COUNTY CSRM 10% DESIGN – Tamiami Trail							
No.	Location		Station	Sill Elevation (ft.)	Elevation of Top of Gate (ft.)	Height (ft.)	Width (ft.)
1	4th Avenue South		3+75	6.6	12.0	5.4	50
2	5th Avenue Parkway		5+75	6.6	12.0	5.4	85
3	(2x) Private Entrance		10+65	3.6	12.0	9.4	25
4	Tamiami Trail (SB)		11+62	2.6	12.0	9.4	50
5	Tamiami Trail (NB)		12+11	2.6	12.0	9.4	70
6	10th Street South		16+50	-0.4	12.0	12.4	65
7	(2x) Private Entrance		20+15	-3.4	12.0	15.4	35
8	Private Entrance		22+40	-4.4	12.0	16.4	25
9	Goodlette Frank Road (SB)		24+95	-6.4	12.0	18.4	65
10	Goodlette Frank Road (NB)		25+50	-6.4	12.0	18.4	65
11	River Point Drive		35+60	-4.4	12.0	16.4	55
12	Private Entrance		47+25	4.6	12.0	7.4	65
13	Private Entrance		51+85	4.6	12.0	7.4	80
14	Davis Boulevard (WB)		55+35	4.6	12.0	7.4	70
15	Davis Boulevard (EB)		56+40	4.6	12.0	7.4	70
16	Entrance to Davis Boulevard		58+00	4.6	12.0	7.4	90
17	Private Entrance		62+65	4.6	12.0	7.4	45
18	(2x) Private Entrance		63+50	4.6	12.0	7.4	40
19	(2x) Private Entrance		65+05	4.6	12.0	7.4	35
20	(2x) Private Entrance		66+30	4.6	12.0	7.4	25
21	(2x) Private Entrance		67+35	4.6	12.0	7.4	40
22	(2x) Private Entrance		69+80	4.6	12.0	7.4	35
23	Private Entrance		71+90	4.6	12.0	7.4	55

PRELIMINARY STREET CLOSURES COLLIER COUNTY CSRM 10% DESIGN – Tamiami Trail							
No.	**Location**		**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
24	Commercial Drive		73+00	4.6	12.0	7.4	80
25	Private Entrance		75+75	4.6	12.0	7.4	40
26	Private Entrance		79+25	4.6	12.0	7.4	25
27	(2x) Private Entrance		79+75	4.6	12.0	7.4	55
28	(2x) Private Entrance		82+90	4.6	12.0	7.4	40
29	Private Entrance		86+35	4.6	12.0	7.4	45
30	Private Entrance		89+50	4.6	12.0	7.4	35
31	Private Entrance		92+35	4.6	12.0	7.4	30
32	Pelton Avenue		94+30	4.6	12.0	7.4	100
33	Private Entrance		96+00	4.6	12.0	7.4	35
34	Pineland Street		98+50	4.6	12.0	7.4	105
35	(2x) Private Entrance		99+75	4.6	12.0	7.4	22
36	(2x) Private Entrance		101+05	4.6	12.0	7.4	25
37	Private Entrance		102+65	4.6	12.0	7.4	20
38	Private Entrance		103+50	4.6	12.0	7.4	40
39	Bayshore Drive		105+10	4.6	12.0	7.4	100
40	Bayside Street		109+35	4.6	12.0	7.4	105
41	Private Entrance		111+10	5.1	12.0	6.9	35
42	Andrew Drive		113+40	5.1	12.0	6.9	75
43	Private Entrance		114+90	5.1	12.0	6.9	30
44	(2x) Private Entrance		116+10	5.1	12.0	6.9	20
45	(2x) Private Entrance		117+15	5.1	12.0	6.9	30
46	Airport Pulling Road South		122+45	5.2	12.0	6.8	175
47	(2x) Private Entrance		127+50	5.2	12.0	6.8	30
48	Private Entrance		129+70	5.2	12.0	6.8	50
49	(2x) Espinal Boulevard		133+95	5.2	12.0	6.8	50
50	Private Entrance		138+15	5.2	12.0	6.8	100

Table 5.23 Bonita Beach Road Alignment Proposed Gate Closures (T.O.W. 10 FT NAVD88)

PRELIMINARY STREET CLOSURES COLLIER COUNTY CSRM 10% DESIGN – Bonita Beach Road							
No.	**Location**		**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
1	Barefoot Beach Boulevard		10+15	2.0	10.0	8.0	150
2	Private Entrance		15+85	1.5	10.0	8.5	40
3	Private Entrance		33+70	1.5	10.0	8.5	60
4	Private Entrance		40+90	3.0	10.0	7	55
5	Private Entrance		48+00	5.0	10.0	5	65
6	Private Entrance		50+20	6.0	10.0	4	40
7	Private Entrance		50+80	6.0	10.0	4	30
8	Private Entrance		52+15	6.0	10.0	4	60
9	Private Entrance		56+00	7.0	10.0	3	50
10	Private Entrance		57+65	7.5	10.0	2.5	30
11	West Avenue		60+25	8.0	10.0	2	110

Table 5.24 Seagate Drive Alignment Proposed Gate Closures (T.O.W. 10 FT NAVD88)

PRELIMINARY STREET CLOSURES COLLIER COUNTY CSRM 10% DESIGN – Seagate Drive						
No.	**Location**	**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
1	Crayon Road	40+65	5.2	10.0	4.8	70

Table 5.25 Tamiami Trail Alignment Proposed Gate Closures (T.O.W. 10 FT NAVD88)

PRELIMINARY STREET CLOSURES COLLIER COUNTY CSRM 10% DESIGN – Tamiami Trail						
No.	**Location**	**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
1	4th Avenue South	3+75	6.6	10.0	3.4	50
2	5th Avenue Parkway	5+75	6.6	10.0	3.4	85
3	(2x) Private Entrance	10+65	3.6	10.0	7.4	25
4	Tamiami Trail (SB)	11+62	2.6	10.0	7.4	50
5	Tamiami Trail (NB)	12+11	2.6	10.0	7.4	70
6	10th Street South	16+50	-0.4	10.0	10.4	65
7	(2x) Private Entrance	20+15	-3.4	10.0	13.4	35
8	Private Entrance	22+40	-4.4	10.0	14.4	25
9	Goodlette Frank Road (SB)	24+95	-6.4	10.0	16.4	65
10	Goodlette Frank Road (NB)	25+50	-6.4	10.0	16.4	65
11	River Point Drive	35+60	-4.4	10.0	14.4	55
12	Private Entrance	47+25	4.6	10.0	5.4	65
13	Private Entrance	51+85	4.6	10.0	5.4	80
14	Davis Boulevard (WB)	55+35	4.6	10.0	5.4	70
15	Davis Boulevard (EB)	56+40	4.6	10.0	5.4	70
16	Entrance to Davis Boulevard	58+00	4.6	10.0	5.4	90
17	Private Entrance	62+65	4.6	10.0	5.4	45
18	(2x) Private Entrance	63+50	4.6	10.0	5.4	40
19	(2x) Private Entrance	65+05	4.6	10.0	5.4	35
20	(2x) Private Entrance	66+30	4.6	10.0	5.4	25
21	(2x) Private Entrance	67+35	4.6	10.0	5.4	40
22	(2x) Private Entrance	69+80	4.6	10.0	5.4	35
23	Private Entrance	71+90	4.6	10.0	5.4	55
24	Commercial Drive	73+00	4.6	10.0	5.4	80
25	Private Entrance	75+75	4.6	10.0	5.4	40
26	Private Entrance	79+25	4.6	10.0	5.4	25
27	(2x) Private Entrance	79+75	4.6	10.0	5.4	55
28	(2x) Private Entrance	82+90	4.6	10.0	5.4	40
29	Private Entrance	86+35	4.6	10.0	5.4	45
30	Private Entrance	89+50	4.6	10.0	5.4	35
31	Private Entrance	92+35	4.6	10.0	5.4	30
32	Pelton Avenue	94+30	4.6	10.0	5.4	100
33	Private Entrance	96+00	4.6	10.0	5.4	35
34	Pineland Street	98+50	4.6	10.0	5.4	105
35	(2x) Private Entrance	99+75	4.6	10.0	5.4	22
36	(2x) Private Entrance	101+05	4.6	10.0	5.4	25
37	Private Entrance	102+65	4.6	10.0	5.4	20
38	Private Entrance	103+50	4.6	10.0	5.4	40
39	Bayshore Drive	105+10	4.6	10.0	5.4	100
40	Bayside Street	109+35	4.6	10.0	5.4	105

PRELIMINARY STREET CLOSURES COLLIER COUNTY CSRM 10% DESIGN – Tamiami Trail							
No.	**Location**		**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
41	Private Entrance		111+10	5.1	10.0	4.9	35
42	Andrew Drive		113+40	5.1	10.0	4.9	75
43	Private Entrance		114+90	5.1	10.0	4.9	30
44	(2x) Private Entrance		116+10	5.1	10.0	4.9	20
45	(2x) Private Entrance		117+15	5.1	10.0	4.9	30
46	Airport Pulling Road South		122+45	5.2	10.0	4.8	175
47	(2x) Private Entrance		127+50	5.2	10.0	4.8	30
48	Private Entrance		129+70	5.2	10.0	4.8	50
49	(2x) Espinal Boulevard		133+95	5.2	10.0	4.8	50
50	Private Entrance		138+15	5.2	10.0	4.8	100

Figure 5.33 Bird's eye view of a gate closure apart of the existing Norfolk Floodwall.

Figure 5.34 A gate closure apart of the existing Norfolk Floodwall.

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5.5.3. STORM SURGE BARRIERS, TIDE GATES, AND SLUICE GATES

Three types of gates were considered for storm surge barriers. They were sector gates (large openings, greater than 100 ft.), barge gates (large openings), and miter gates (smaller openings, less than 100 ft.). Sluice gates (tide gates) were also considered in conjunction with miter gates for areas with wider openings and in one area with existing drainage culverts. Hydrodynamic modeling, to analyze the potential impacts of the flow through the proposed barriers and potential impacts to water quality, has not yet been performed on the proposed gate configurations for this study. Therefore, gate configurations were determined by considering past hydrodynamic modeling reports on the channels provided by the county.

5.5.3.1. SECTOR GATES

Sector gates are shaped like a slice of pie with a triangular framework making up the majority of the gate and a solid skin plate that wraps around the outer curved edge. Sector gates were considered for crossing Wiggins Pass and Doctors Pass. Based upon the navigable channel width of 150 feet (at the toe), a 150 foot wide sector gate was examined as an option to cross the channel. The top of gate height was preliminarily estimated to be at Elevation 11.0 NAVD 88 and the bottom of the gate at -12.0 NAVD 88. These elevations were selected in consideration of equipment systems requirements and potential scour or accretion. The “go by” for estimating purposes was “New Bedford – Fairhaven Barrier” (**Figure 5.35**) located in the New England

District. This gate was selected as its geometry and opening width was similar to the one we anticipate for the Collier project. Sector gates have the advantage of relatively easy and fast opening/closing. They can span great widths and remain partially open for extended periods of time if needed. The main disadvantages are the large footprint of the structure itself and the significant cost of construction. **Figure 5.36** shows a USACE New Orleans District Sector Gate in construction. This gate is the Bayou Dupre Sector Gate.

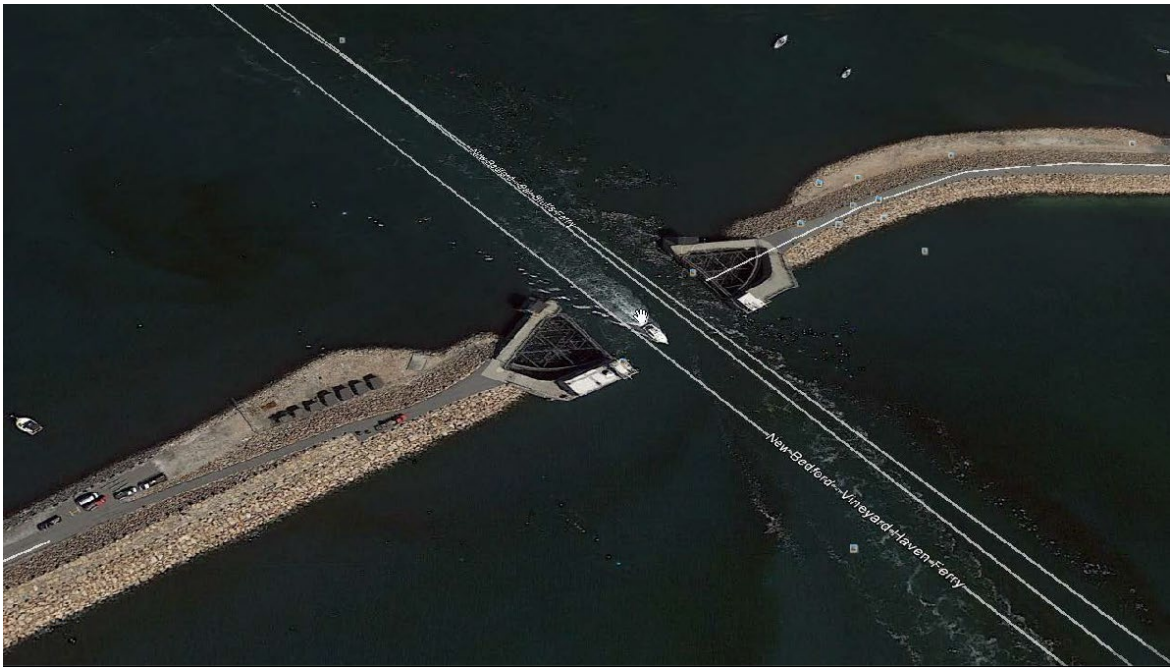

Figure 5.35 Bird's Eye View of the New Bedford – Fairhaven Barrier

Figure 5.36 Bayou Dupre Sector Gate under construction.

5.5.3.2. BARGE GATES

Barge Gates are box shaped, buoyant, hydraulic closures that can be floated (depending on if the barge is not fixed/attached to the closure) or swung (if the barge is fixed to the closure) into position and sunk by filling with water, sand, stones, or etc. Barge gates were considered due to

the ability to provide a continuous 150 foot opening at Wiggins Pass without any obstruction. The main advantage of barge gates is the cost. Compared to the other two types considered, barge gates are the most economical. However, further research revealed a myriad of factors that gave the engineering team pause on using this type of gate. It was found that closures could take anywhere from 4 to 12 hours due to the sill having to be cleaned of debris in order for the barge to form a tight seal. Additionally, the barge gates were eliminated because of the shallow rock found in this area. Barge gate abutments/walls are comprised of sheet piles which cannot be driven into rock without damage. Further geotechnical exploration may be needed to determine the feasibility of barge gates at Wiggins Pass and Doctors Pass. Figure 5.37 shows the Bayou Petit Caillou Barge Gate for reference.

Figure 5.37 Image of the Bayou Petit Caillou Barge Gate

5.5.3.3. MITER GATES

Storm surge barriers that crossed river inlets were determined to be miter gates. The miter gate system was selected as it is commonly employed in these types of projects. Miter Gates consist of a pair of gates, anchored to reinforced concrete abutments at either river bank, that swing out and meet at an angle pointing toward the upstream direction. The gate is kept closed through a difference in water levels. The upstream side should have higher hydraulic loading as the water level rises thus strengthening the locking effect. The angled point reduces any leaks from between the gate doors and prevents their opening until water levels have equalized. Miter gates spanning across openings of 40 to 75 feet are used at the Richmond Filtration Plant flood

mitigation project (**Figure 5.38)** and the Atlantic Intercostal Waterway. The miter gate barriers for this project would be constructed at river passages to block the flow of storm surge during a storm event. The miter gate spans are commonly 70 feet in width and the depths range between Elevation 0.0 NAVD 88 to -7.0 NAVD 88 feet. All of these dimensions work well with the miter gate designs we currently use. Cost estimates are based on existing gate constructions and operations, making adjustments to suit the Collier CSRM study.

Figure 5.38 Image of Miter Gate at Richmond Filtration Plant flood Mitigation Project

5.5.3.4. SLUICE GATES

Tide gates or barriers that cross small tidal creeks or at drainage structures through a levee or floodwall are proposed to be sluice gates. The sluice gates considered for this study are vertical rising sluice gates which are usually metal plates, typically controlled by machinery. Norfolk District has extensive experience using vertical rising sluice gates on several existing flood mitigation projects. **Figure 5.39** is an example of sluice gate through a levee.

Figure 5.39 Image of a Sluice Gate through a levee.

5.5.3.5. FLAP GATES

Flap gates (Figure 5.40) permit the free flow of water from a pipe or structure while preventing and sealing against backflow and allow water to flow in one direction only through a culvert. Flap gates may be proposed to be added to all storm drains that drain out into the open waterways to prevent water from high tides from backing up the storm drainage systems.

Figure 5.40 Image of a Flap Gate at the end of a culvert.

5.5.3.6. MITER/SLUICE COMBINATION GATES

A combination gate is proposed at Wiggins Pass which is comprised of a 72 foot wide miter gate with two 40 foot wide sluice gates on either side. A 150 foot wide miter gate was considered however, the maximum structurally economic width of a miter gate was determined to be 72 feet. Therefore a miter/sluice combination gate was proposed in order to accommodate a 150 foot wide channel while being reasonably economical. All three gates would be kept opened during non-flood conditions for water flow with the miter gate being the only one open to navigational traffic. The main advantages of this type of gate are the reliability, relatively fast opening/closing times, and low footprint compared to sector and barge gates. The main disadvantage was that this gate would not be able to provide a continuously open 150 foot wide channel due to the need for the gate abutments in between. See **Figure 5.42** for a schematic of the proposed configuration. **Figure 5.42** shows a sector/sluice combination gate of the Seabrook Floodgate Complex in New Orleans, LA for reference. The sector gate would instead be a miter gate.

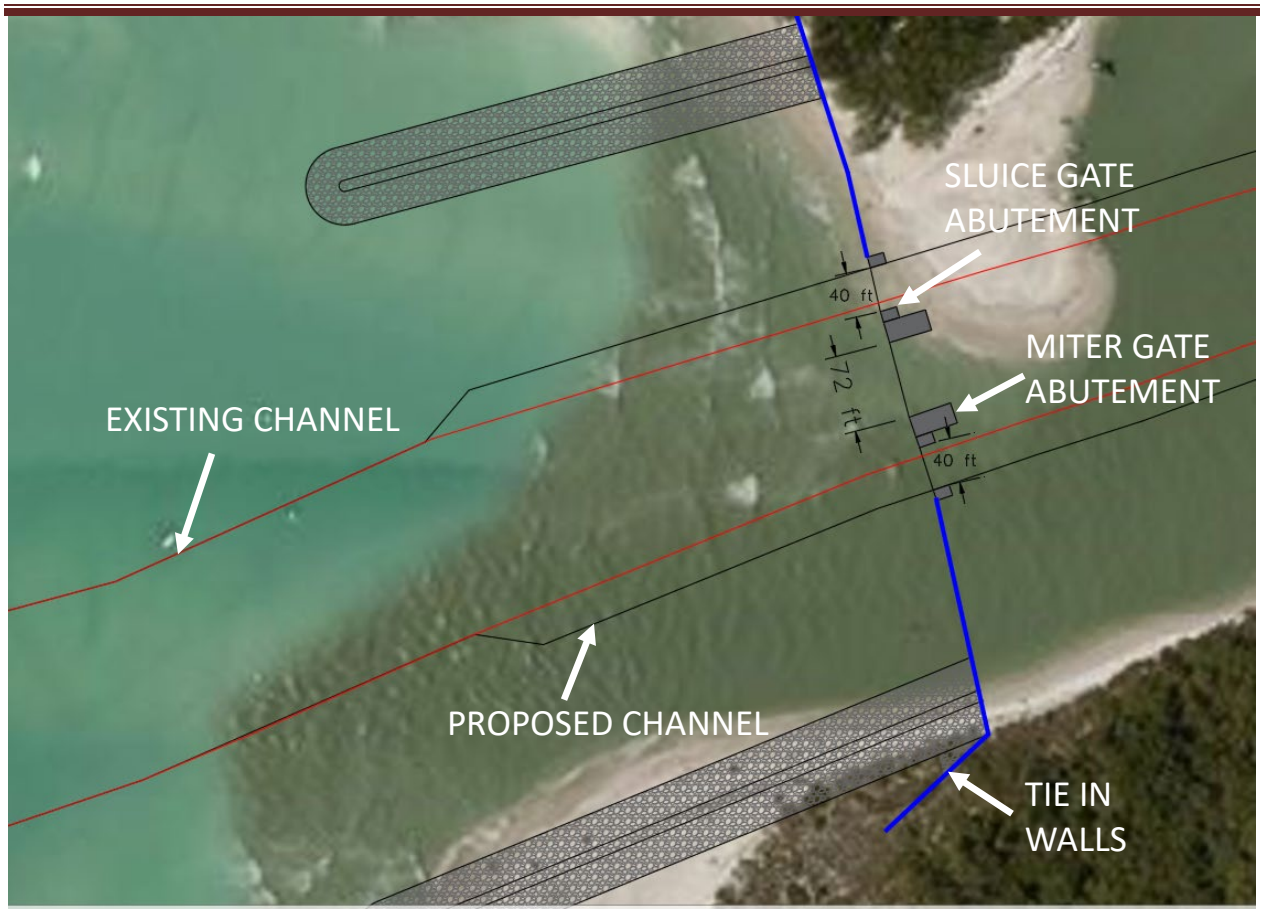

Figure 5.41 Wiggins Pass Miter/Sluice Combination Gate Schematic

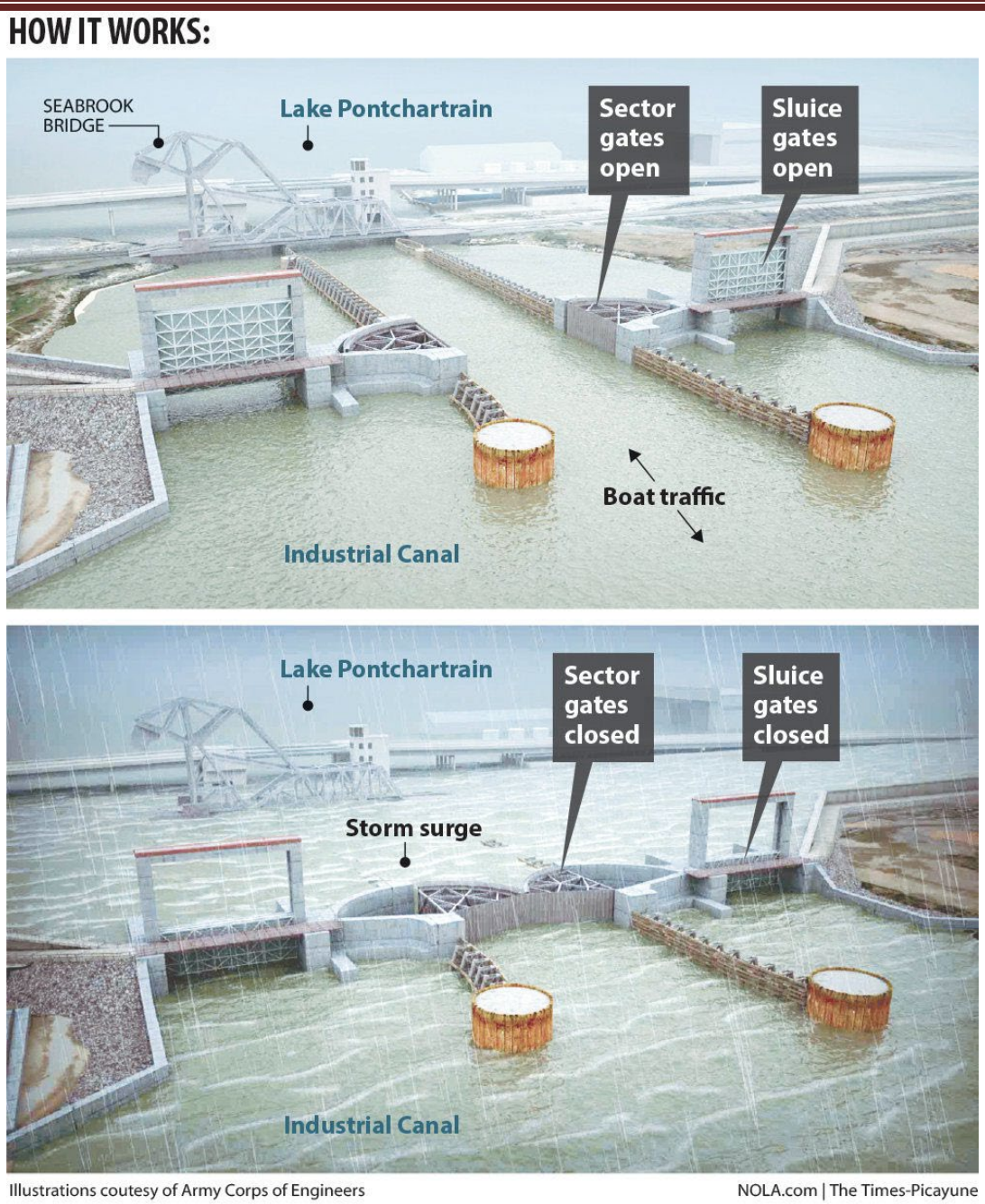

Figure 5.42 Seabrook Floodgate Complex, New Orleans, LA.

5.5.4. WAVE FORCES ON VERTICAL WALL

The Goda method was used to calculate wave forces on a vertical wall. The Goda method was used because the equations for this method are applicable to nonbreaking, breaking, and broken waves, no distinctions are made between the three. The formulas give additional force due to the waves, so that the still-water hydrostatic force must be added to give the total forces on the wall. The wave forces calculated for this study were provided to the structural and geotechnical engineers for their analysis and calculation of the total forces on I-walls and T-

walls. More information on the calculations and how they were applied in the study can be read in the Hydraulics, Hydrology, & Coastal, Geotechnical and Structural sub-appendices.

5.5.5. WAVE OVERTOPPING

Wave overtopping was analyzed using the FEMA SWFL Study water levels and wave heights. The interactive computer-based design and analysis system, ACES (which is based on equations found in the Corps' of Engineers Coastal Engineering Manual (CEM)) was used for this analysis. The overtopping analysis, for the designed vertical wall and levee, used the interactive computer-based design and analysis system, ACES (which is based on equations found in the Corps' of Engineers Coastal Engineering Manual (CEM)) and the online version of EurOtop (which is based off equations that can be found in the EurOtop Overtopping Manual) for comparison. The analysis included the development of peak overtopping rates for the 5% Flood (20 year), 2% Flood (50 year), and 1% Flood (100 year). ACES and EurOtop used both the Probabilistic and Deterministic approaches to analyze the overtopping for the design of the vertical wall and levee.

5.5.6. GENERATION II COASTAL RISK MODEL (G2CRM) ANALYSIS

The Generation II Coastal Risk Model (G2CRM) was developed to support planning-level studies of hurricane protection systems (HPS). The G2CRM is a desktop computer model that implements an object-oriented probabilistic life cycle analysis (PLCA) model using event-driven Monte Carlo simulation (MCS). This allows for incorporation of time-dependent and stochastic event-dependent behaviors such as sea level change, tide, and structure raising and removal. The model is based upon driving forces (storms) that affect a coastal region (study area). The study area is comprised of individual sub-areas of different types that may interact hydraulically and may be defended by coastal defense elements that serve to shield the areas and the assets they contain from storm damage. The model is scalable in that different levels of detail can be used for the data that drives the model, with lower levels of detail at early stages of model application (fewer storms, aggregated assets) and more refined representations used as new data become available (USACE 2018). See the Economics Appendix for more information regarding the development of the G2CRM model areas and asset inputs.

5.5.6.1. METEROLOGICAL DRIVING FORCES

Similar to Beach-*fx*, G2CRM requires a probabilistic storm suite (storm hydrographs (surge and waves) at locations, as generated externally from high fidelity storm surge and nearshore wave models such as ADCIRC and STWAVE) as the driving forces. For Collier County, all 357 storms from the FEMA SWFL Study data were included in the G2CRM Tropical Storm probabilistic storm suite. The water level and wave hydrographs provided from these storms were developed into an h5 input file to be used by the model using the data from various save points. Though G2CRM is capable of generating its own tidal forcings, random phase tides were already embedded in the data provided by FEMA and thus also included in the processed storm hydrographs provided from ERDC. In terms of G2CRM this mean the model was run with tides "turned-off" in the model so as to not add additional tide data during model runs. While this may

potentially limit the number of external tidal forcings available in the model, the data was deemed sufficient to make planning level decisions for the project area. Future analysis during the PED phase should use updated storms with more tidal variation on TSP beaches reaches to refine and further develop the model.

Extra-Tropical storms were not included in the G2CRM storm suite. The available Extra-Tropical storms included in the Beach-*fx* storm suite were for an offshore point and deemed not suitable for use with the back bay interior save points as the waves in the back bay would be more sheltered than those offshore and thus damage could be overestimated. The water levels for the Extra-Tropical storms are low, generally less than 2 ft NAVD88, and minimal to no damage would be anticipated from these storms in the back bay and therefore it was deemed suitable to exclude from the G2CRM storm suite.

In addition to the plausible storm dataset, the seasonality of the storms must also be specified. The Collier County G2CRM model utilized the same storm seasons as the Beach-*fx* model (see Table 5.7).

5.5.6.2. MODELING AREAS

The G2CRM model is separated into different model study areas, each with its own h5 file associated with that model area to provide the environmental forcings during the life cycle simulation. Each h5 file location corresponds to a save point within the model area that was determined to have the most appropriate forcings (water level and wave storm hydrographs) for that area. Back bay flooding for Beach-*fx* model reaches were modeled with G2CRM in addition to more inland back bay areas.

See the below Figure 5.43 to Figure 5.45 and Table 5.26 for the locations of the Collier County model areas and the locations for the save point processed for the associated h5 files. The boundaries of the larger model areas protected by structural measures (MA24 and 28) is the 12 ft contour from the DEM. The structures within these model areas are also shown and where relevant, the Beach-*fx* model reaches are also labeled. A few model areas share the same save point, however one is designated at “no waves.” For these model areas (MA 24b, MA24d, and MA28b), it was decided to remove the wave hydrographs in order to not over estimate damages in upland areas far from the save point and it is expected that wave influences as a surge moves inland will be minimal to null. Therefore, the damage in these "no wave" areas will only depend on the water level hydrographs. It is expected that increasing first floor elevations of the structures in areas of higher elevations near the boundaries will result in fewer damages due to the water levels.

In order to not double count damages, the damages elements in the beach model reaches will be compared between G2CRM and Beach-*fx* results for the highest damage level and only model producing the highest results will be used. i.e. if back bay flooding causes more damage than beach overtopping flooding, the G2CRM results for that damage element will be used to calculate benefits. For more information on the set up of model areas and structures in G2CRM

and how benefits are calculated see the Economics Appendix.

Table 5.26 Collier County Model Areas and Save Points

Planning Area	Save Point (for h5 file)	Model Area	Associated Beach-*fx* Model Reaches If Applicable (Table 5.3)
1	192	MA01a	BB1
	174	MA02a	BB2, WP, VB1, VB2
	192	MA23a	
	171	MA24a	
	171 (no waves)	MA24b	
	155	MA24c	
	155 (no waves)	MA24d	
2	143	MA06a	PB1, PB2, CP
	143	MA25a	
3	121	MA09a	PS1
	108	MA10a	PS2a, PS2b
	229	MA12a	NB1, NB2, NB3, NB4
	121	MA26a	
	108	MA27a	
4	72	MA16a	NB5. NB6
	68	MA18a	NB7, GP1, GP2, GP3, GP4
	68	MA29a	
5	209	MA28a	
	209 (no waves)	MA28b	
6	44	MA30a	
	37	MA30b	
	34	MA31NW	
	37	MA31N	
	28	MA31Ea	
	41	MA31Eb	
	20	MA31WS	
	7	MA31ES	
	8	MA31S	
	20	MA32a	MI1, MI2a, MI2b, MI2c, MI3, MI4a, MI4b
	1	MA39a	

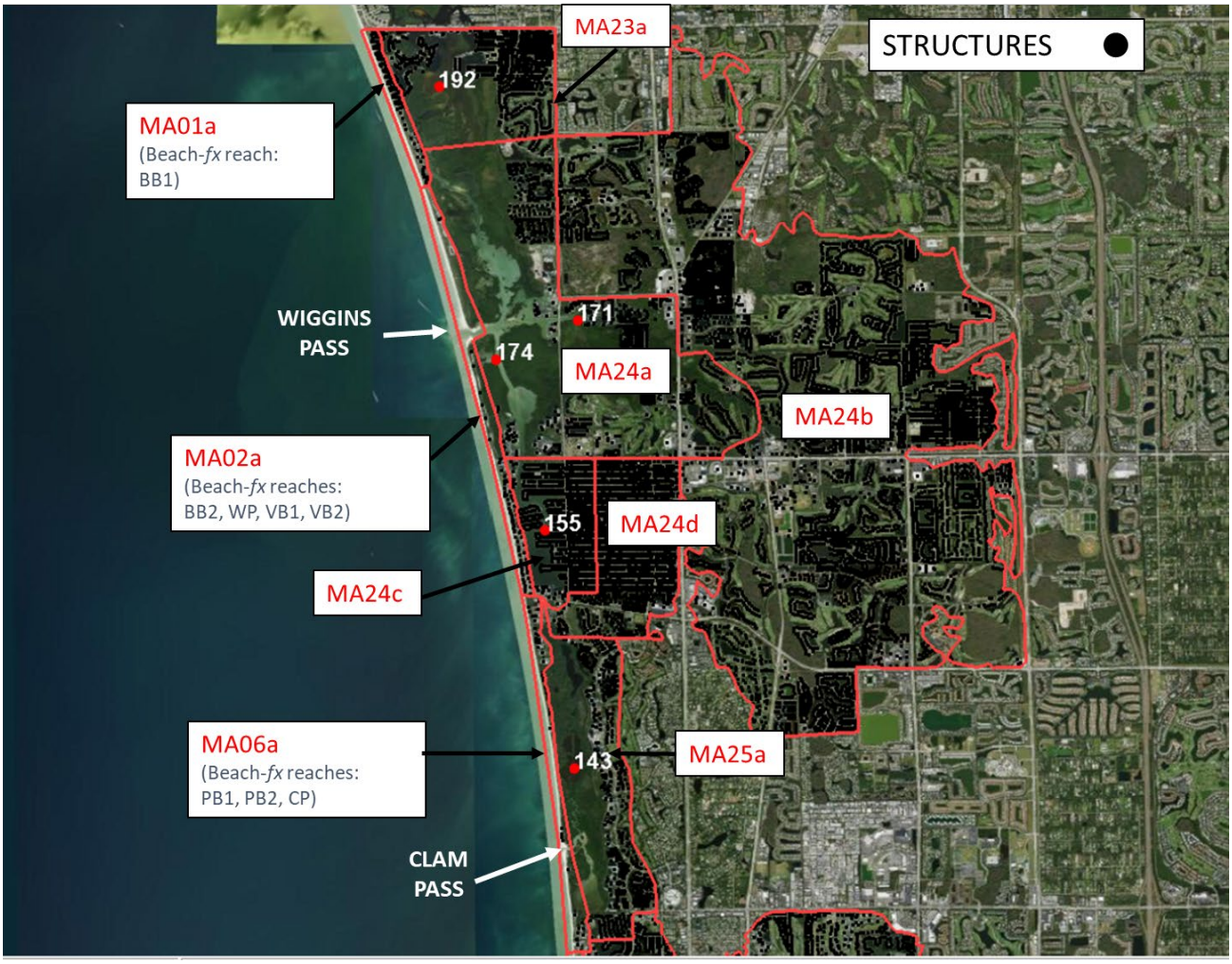

Figure 5.43 Collier County G2CRM Model Areas and Save Points- Bonita Beach Rd. to Seagate Dr.

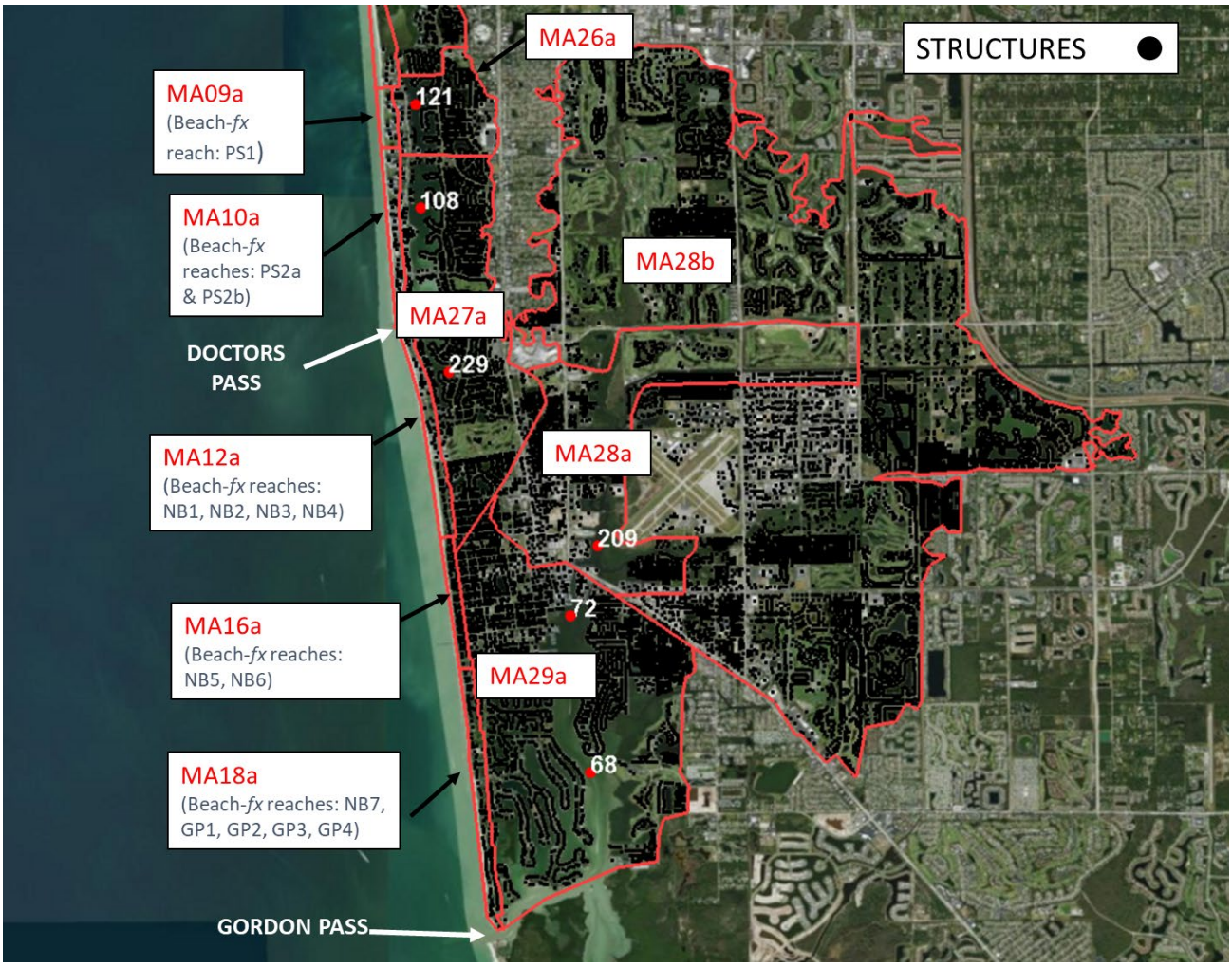

Figure 5.44 Collier County G2CRM Model Areas and Save Points- Seagate Dr. to Gordon Pass

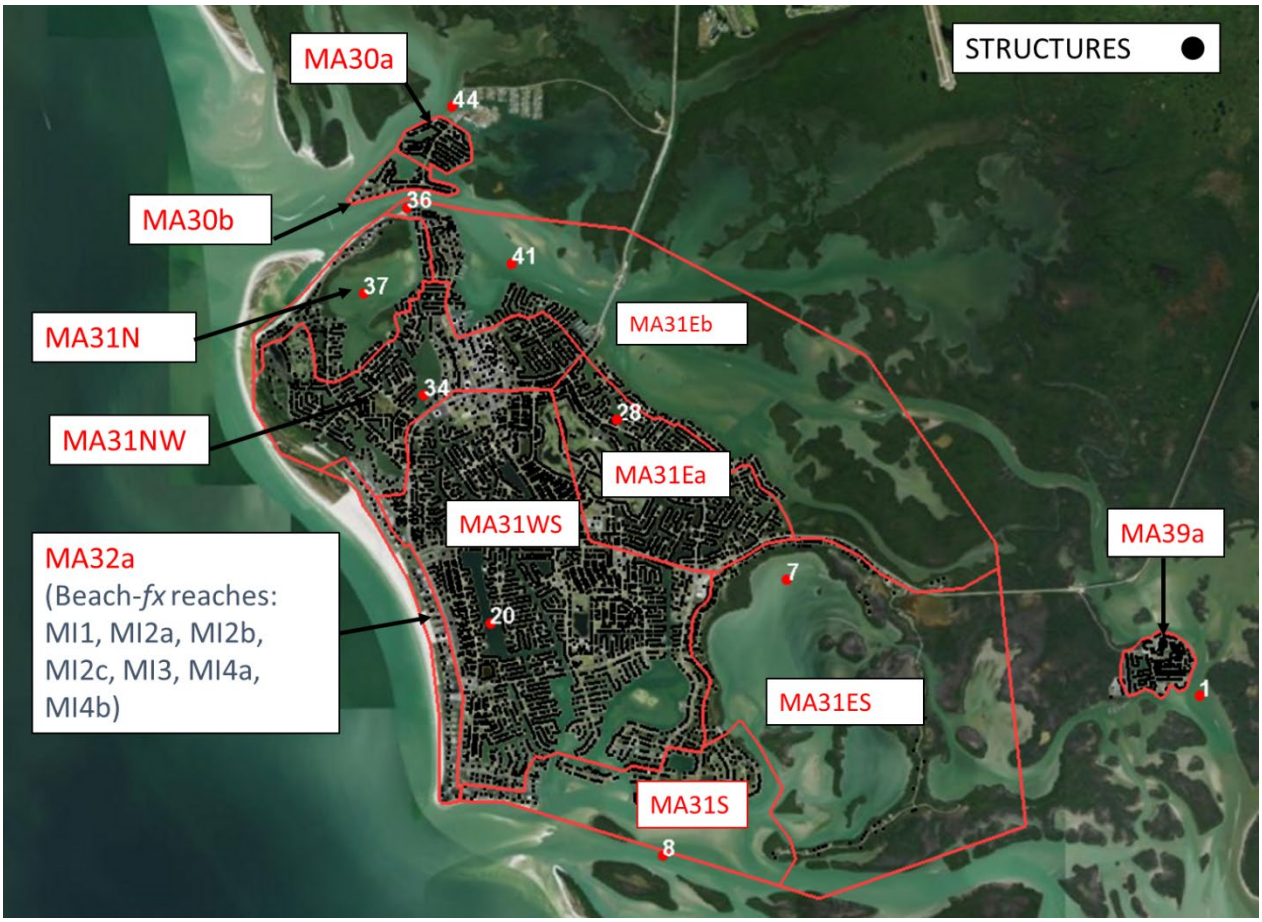

Figure 5.45 Collier County G2CRM Model Areas and Save Points- Marco Island

5.5.6.3. PROTECTIVE SYSTEM ELEMENTS

G2CRM utilizes Protective System Elements (PSEs) as the infrastructure that defines the coastal boundary be it a coastal defense system that protects the modeled areas from flooding (levees, pumps, closure structures, etc.), or a locally developed coastal boundary comprised of bulkheads and/or hardened shoreline. These structures are set at heights specified by the user and the model is then run to estimate the effectiveness of the structure against preventing damages. The heights used for the various Collier County Protective elements (surge barriers and floodwalls) were initially developed by considering the overtopping analysis performed at each structure for various levels of protection (Paragraph 5.5.5 WAVE OVERTOPPING in this Appendix) and then were refined based on the sensitivity of these heights to sea level rise and for optimizing cost/benefit relationship. See the Economics Appendix for more information regarding the G2CRM model and analysis.

5.5.7. ECOLOGICAL/ENVIRONMENTAL IMPACTS

Hydrodynamic modeling was completed for water quality analysis and environmental impacts.

The quality modeling effort is discussed in the Environmental Appendix.

*To include short summary when modeling is complete.

5.6. INTERIOR FLOODING ANALYSIS

According to Engineering Manual 1110-2-1413, an interior area is defined as the area protected from direct riverine, lake, or tidal flooding by levees, floodwalls or seawalls and low depression or natural sinks. Management measures, such as a levee or wall, associated with an interior area is generally referred to as the project alignment. The project alignment excludes flood water originating from the exterior but normally does not directly alleviate flooding that may subsequently occur from interior rainfall runoff. In fact, the project alignment can often aggravate the problem of interior flooding by blocking drainage outlets. For this study, the Collier County Watershed Management Plan and South Florida Water Management District reports and GIS layers were used to assess the need for interior drainage features such as tidal sluice gates and pumps. Further information on the interior flooding analysis can be found in Hydraulics, Hydrology and Coastal Sub-Appendix in Chapter 7.

5.7. QUANTITY ESTIMATES

The following tables provides the quantities for each structural measure that was determined from the different project alignments and for the proposed Tentatively Selected Plan (TSP). The TSP is a hybrid plan that includes structural measures, where they were found to be feasible and justified, and nonstructural measures elsewhere to provide greater coverage of the study area and further buy-down residual risk. The quantities in the table below only include structural measures, see the Economics Appendix for non-structural quantities and costs are discussed. These quantities are subject to change with optimized model runs.

Table 5.27 Project Alignment Quantities (10 FT NAVD88)

AREA	ELEMENTS OF MEASURES	Planning Area	LOCATION	ASSUMED AVG GRADE ELEV . (FT)	TOP OF WALL/DUNE/ CREST ELEV. (FT)	WALL TYPE	# STREET CLOSURES	APPROX. LENGTH OF PROPOSED STRUCTURES (FT)	APPROX. VOLUME OF PROPOSED STRUCTURES
Barefoot Beach	1	1	West Bonita Beach Rd. from dune to East Tidal Barrier	3.5	10	T-WALL	2	2,750	11,317 CY
			Length of East Tidal Barrier - 100' Miter Gate ; Barrier Along Bridge						
			Bonita Beach Rd near Barefoot Boat Club between East and Wast Tidal Barriers	9.5	10	T-WALL	2	1,750	
			Length of West Tidal Barrier - 100' Miter Gate ; Barrier Along Bridge						
			East Bonita Beach Rd. west of West Tidal Barrier	9.5	10	T-WALL	7	1,470	
	2		Beach Berm and Dune	3.5	10			8,110	10 FT dune, 75 ft berm Initial Construction: 572,600 CY Each Renourishment: 81,400 CY
Barefoot Beach Preserve	3	1	Beach Berm and Dune	3.5	10			7,475	10 FT dune, 75 ft berm Initial Construction: 81,400 CY Each Renourishment: 262,371 CY
Wiggins Pass State Park	4	1	Northern Jetty		10			430	Armor Rock- 23,100 TONS Bedding Rock- 3,280 TONS Filter cloth- 39,800 SF Geogrid - 39,800 SF
			Northern Barrier Wall Tie - In	-1.6	10	T-WALL	0	245	2,105 CY
			Wiggins Pass Tidal Barrier - 150'. Combo wall with Miter gates and Lift gate at navigation channel.						
			Southern Barrier Wall Tie - In	-3.3	10	T-WALL	0	270	
			Southern Jetty		10			575	Armor Rock- 22,700 TONS Bedding Rock- 3,830 TONS Filter cloth- 39,800 SF Geogrid- 39,800 SF
	5		Beach Berm and Dune	3.5	10			6,125	10 FT dune, 75 ft berm Initial Construction: 208,500 CY Each Renourishment: 382,771 CY
Vanderbilt	6	1	Beach Berm and Dune	3.5	12			6,900	10 FT dune, 75 ft berm Initial Construction: 227,600 CY Each Renourishment: 259,657 CY
Pelican Bay	7	2	Beach Berm and Dune	3.5	10			12,150	10 FT dune, 75 ft berm Initial Construction: 396,400 CY Each Renourishment: 450,329 CY
Clam Pass State Park	8	2	Beach Berm and Dune	3.5	10			5,440	10 FT dune, 75 ft berm Initial Construction: 362,200 CY Each Renourishment: 219,547 CY
Park Shore	9	3	West Seagate Dr from beach dune to Sluice Gates	2.5	10	T-WALL	0	1,050	7,984 CY
			(3x) Sluice Gate 19' x 24" (Inside Dia.)						
			East Seagate Dr.	6.6	10	T-WALL	1	3,450	
	10		Beach Berm and Dune	4.5/3.5	10			11,330	10 FT dune, 75 ft berm Initial Construction: 343,600 CY Each Renourishment: 284,629 CY
Naples	11	3	Northern Barrier Wall Tie - In	2.9	10	T-WALL	0	120	891 CY
			Doctors Pass Tidal Barrier -Length 100'. Miter gates at navigation channel.						
			Southern Barrier Wall Tie - In	1.65	10	T-WALL	0	125	
	12	5	East Tamiami Trail E	4	10	T-WALL	10	2,700	16,298 CY
	13		Tamiami Trail E East Tie-In	5.5	10	T-WALL	0	85	
			Length of East Tidal Barrier - 72'; Barrier Along Bridge						
			Naples Island East Barrier Tie-In	2	10	T-WALL	0	70	
	14		Naples Island	-2	10	T-WALL	1	935	
	15		Naples Island West Barrier Tie-In	-6	10	T-WALL	0	70	
			Length of West Tidal Barrier - 72'; Barrier Along Bridge						
			Tamiami Trail E West Tie-In	6.5	10	T-WALL	0	65	
	16		West Tamiami Trail E	6.8	10	T-WALL	39	9,925	
	17	3	Beach Berm and Dune	3.5	10			10,611	10 FT dune, 75 ft berm Initial Construction: 728,500 CY Each Renourishment: 525,157 CY
	18	4	Beach Berm and Dune	3.5	10			9,475	10 FT dune, 75 ft berm Initial Construction: 616,500 CY Each Renourishment: 269,657 CY
Gordon Pass	19	4	Beach Berm and Dune	3.5	10			9,660	10 FT dune, 75 ft berm Initial Construction: 616,500 CY Each Renourishment: 484,800 CY
Marco Island	20	6	South Marco Island Revetment Improvement		10			3,590	Armor Rock- 124,150 TONS Filter cloth- 256,900 SF

Table 5.28 Structural Project Alignment Quantities (12 FT NAVD88)

AREA	ELEMENTS OF MEASURES	Planning Area	LOCATION	ASSUMED AVG GRADE ELEV . (FT)	TOP OF WALL/DUNE/ CREST ELEV. (FT)	WALL TYPE	# STREET CLOSURES	APPROX. LENGTH OF PROPOSED STRUCTURES (FT)	APPROX. VOLUME OF PROPOSED STRUCTURES
Barefoot Beach	1	1	West Bonita Beach Rd. from dune to East Tidal Barrier	3.5	12	T-WALL	2	2,750	12,028 CY/TON
			Length of East Tidal Barrier - 100' Miter Gate ; Barrier Along Bridge						
			Bonita Beach Rd near Barefoot Boat Club between East and Wast Tidal Barriers	9.5	12	T-WALL	2	1,750	
			Length of West Tidal Barrier - 100' Miter Gate ; Barrier Along Bridge						
			East Bonita Beach Rd. west of West Tidal Barrier	9.5	12	T-WALL	7	1,470	
	2		Beach Berm and Dune	3.5	12			8,110	12 FT dune, 75 ft berm Initial Construction: 749,800 CY Each Renourishment: 205,714 CY
Barefoot Beach Preserve	3	1	Beach Berm and Dune	3.5	12			7,475	12 FT dune, 75 ft berm Initial Construction: 41,400 CY Each Renourishment: 274,300 CY
Wiggins Pass State Park	4	1	Northern Jetty		12			430	Armor Rock- 23,100 TONS Bedding Rock- 3,280 TONS Filter cloth- 39,800 SF Geogrid - 39,800 SF
			Northern Barrier Wall Tie - In	-1.6	12	T-WALL	0	245	2,194 CY
			Wiggins Pass Tidal Barrier - 150'. Combo wall with Miter gates and Lift gate at navigation channel.						
			Southern Barrier Wall Tie - In	-3.3	12	T-WALL	0	270	
			Southern Jetty		12			575	Armor Rock- 22,700 TONS Bedding Rock- 3,830 TONS Filter cloth- 39,800 SF Geogrid- 39,800 SF
	5		Beach Berm and Dune	3.5	12			6,125	12 FT dune, 75 ft berm Initial Construction: 343,300 CY Each Renourishment: 398,271 CY
Vanderbilt	6	1	Beach Berm and Dune	3.5	12			6,900	10 FT dune, 75 ft berm Initial Construction: 378,100 CY Each Renourishment: 272,729 CY
Pelican Bay	7	2	Beach Berm and Dune	3.5	12			12,150	12 FT dune, 75 ft berm Initial Construction: 662,800 CY Each Renourishment: 471,657 CY
Clam Pass State Park	8	2	Beach Berm and Dune	3.5	12			5,440	12 FT dune, 75 ft berm Initial Construction: 484,800 CY Each Renourishment: 225,786 CY
Park Shore	9	3	West Seagate Dr from beach dune to Sluice Gates	2.5	12	T-WALL	0	1,050	13,711 CY
			(3x) Sluice Gate 19' x 24" (Inside Dia.)						
			East Seagate Dr.	6.6	12	T-WALL	1	3,450	
	10		Beach Berm and Dune	4.5/3.5	12			11,330	12 FT dune, 75 ft berm Initial Construction: 593,000 CY Each Renourishment: 300,457 CY
Naples	11	3	Northern Barrier Wall Tie - In	2.9	12	T-WALL	0	120	1,615 CY
			Doctors Pass Tidal Barrier -Length 100'. Miter gates at navigation channel.						
			Southern Barrier Wall Tie - In	1.65	12	T-WALL	0	125	
	12	5	East Tamiami Trail E	4	12	T-WALL	10	2,700	22,009 CY
	13		Tamiami Trail E East Tie-In	5.5	12	T-WALL	0	85	
			Length of East Tidal Barrier - 72'; Barrier Along Bridge						
			Naples Island East Barrier Tie-In	2	12	T-WALL	0	70	
	14		Naples Island	-2	12	T-WALL	1	935	
	15		Naples Island West Barrier Tie-In	-6	12	T-WALL	0	70	
			Length of West Tidal Barrier - 72'; Barrier Along Bridge						
			Tamiami Trail E West Tie-In	6.5	12	T-WALL	0	65	
	16		West Tamiami Trail E	6.8	12	T-WALL	39	9,925	
	17	3	Beach Berm and Dune	3.5	12			10,611	12 FT dune, 75 ft berm Initial Construction: 978,500 CY Each Renourishment: 546,071 CY
	18	4	Beach Berm and Dune	3.5	12			9,475	12 FT dune, 75 ft berm Initial Construction: 837,500 CY Each Renourishment: 282,014 CY
Gordon Pass	19	4	Beach Berm and Dune	3.5	12			9,660	12 FT dune, 75 ft berm Initial Construction: 696,000 CY Each Renourishment: 324,486 CY
Marco Island	20	6	South Marco Island Revetment Improvement		12			3,590	Armor Rock- 146,055 TONS Filter cloth- 256,900 SF

Table 5.29 Structural Project Alignment Quantities (14 FT NAVD88)

AREA	ELEMENTS OF MEASURES	Planning Area	LOCATION	ASSUMED AVG GRADE ELEV . (FT)	TOP OF WALL/DUNE/ CREST ELEV. (FT)	WALL TYPE	# STREET CLOSURES	APPROX. LENGTH OF PROPOSED STRUCTURES (FT)	APPROX. VOLUME OF PROPOSED STRUCTURES
Barefoot Beach	1	1	West Bonita Beach Rd. from dune to East Tidal Barrier	3.5	14	T-WALL	2	2,750	12,739 CY
			Length of East Tidal Barrier - 100' Miter Gate ; Barrier Along Bridge						
			Bonita Beach Rd near Barefoot Boat Club between East and Wast Tidal Barriers	9.5	14	T-WALL	2	1,750	
			Length of West Tidal Barrier - 100' Miter Gate ; Barrier Along Bridge						
			East Bonita Beach Rd. west of West Tidal Barrier	9.5	14	T-WALL	7	1,470	
	2		Beach Berm and Dune	3.5	14			8,110	14 FT dune, 75 ft berm Initial Construction: 939,800 CY Each Renourishment: 224,786 CY
Barefoot Beach Preserve	3	1	Beach Berm and Dune	3.5	14			7,475	14 FT dune, 75 ft berm Initial Construction: 199,800 CY Each Renourishment: 290,500 CY
Wiggins Pass State Park	4	1	Northern Jetty		14			430	Armor Rock- 23,100 TONS Bedding Rock- 3,280 TONS Filter cloth- 39,800 SF Geogrid - 39,800 SF
			Northern Barrier Wall Tie - In	-1.6	14	T-WALL	0	245	2,283 CY
			Wiggins Pass Tidal Barrier - 150'. Combo wall with Miter gates and Lift gate at navigation channel.						
			Southern Barrier Wall Tie - In	-3.3	14	T-WALL	0	270	
			Southern Jetty		14			575	Armor Rock- 22,700 TONS Bedding Rock- 3,830 TONS Filter cloth- 39,800 SF Geogrid- 39,800 SF
	5		Beach Berm and Dune	3.5	14			6,125	14 FT dune, 75 ft berm Initial Construction: 490,900 CY Each Renourishment: 421,157 CY
Vanderbilt	6	1	Beach Berm and Dune	3.5	12			6,900	14 FT dune, 75 ft berm Initial Construction: 544,000 CY Each Renourishment: 285,243 CY
Pelican Bay	7	2	Beach Berm and Dune	3.5	14			12,150	14 FT dune, 75 ft berm Initial Construction: 956,900 CY Each Renourishment: 495,643 CY
Clam Pass State Park	8	2	Beach Berm and Dune	3.5	14			5,440	14 FT dune, 75 ft berm Initial Construction: 681,800 CY Each Renourishment: 236,657 CY
Park Shore	9	3	West Seagate Dr from beach dune to Sluice Gates	2.5	14	T-WALL	0	1,050	15,245 CY
			(3x) Sluice Gate 19' x 24" (Inside Dia.)						
			East Seagate Dr.	6.6	14	T-WALL	1	3,450	
	10		Beach Berm and Dune	4.5/3.5	14			11,330	14 FT dune, 75 ft berm Initial Construction: 859,800 CY Each Renourishment: 318,557 CY
Naples	11	3	Northern Barrier Wall Tie - In	2.9	14	T-WALL	0	120	1,686 CY
			Doctors Pass Tidal Barrier -Length 100'. Miter gates at navigation channel.						
			Southern Barrier Wall Tie - In	1.65	14	T-WALL	0	125	
	12	5	East Tamiami Trail E	4	14	T-WALL	10	2,700	28,814 CY
	13		Tamiami Trail E East Tie-In	5.5	14	T-WALL	0	85	
			Length of East Tidal Barrier - 72'; Barrier Along Bridge						
			Naples Island East Barrier Tie-In	2	14	T-WALL	0	70	
	14		Naples Island	-2	14	T-WALL	1	935	
	15		Naples Island West Barrier Tie-In	-6	14	T-WALL	0	70	
			Length of West Tidal Barrier - 72'; Barrier Along Bridge						
			Tamiami Trail E West Tie-In	6.5	14	T-WALL	0	65	
	16		West Tamiami Trail E	6.8	14	T-WALL	39	9,925	
	17	3	Beach Berm and Dune	3.5	14			10,611	14 FT dune, 75 ft berm Initial Construction: 1,248,800 CY Each Renourishment: 577,886 CY
	18	4	Beach Berm and Dune	3.5	14			9,475	14 FT dune, 75 ft berm Initial Construction: 1,083,000 CY Each Renourishment: 298,743 CY
Gordon Pass	19	4	Beach Berm and Dune	3.5	14			9,660	14 FT dune, 75 ft berm Initial Construction: 931,700 CY Each Renourishment: 331,671 CY
Marco Island	20	6	South Marco Island Revetment Improvement		14			3,590	Armor Rock- 157,290 TONS Filter cloth- 256,900 SF

Table 5.30 Structural Project Alignment Quantities (TSP)

AREA	ELEMENTS OF MEASURES	Planning Area	LOCATION	ASSUMED AVG GRADE ELEV . (FT)	TOP OF WALL/DUNE/ CREST ELEV. (FT)	WALL TYPE	# STREET CLOSURES	APPROX. LENGTH OF PROPOSED STRUCTURES (FT)	APPROX. VOLUME OF PROPOSED STRUCTURES
Barefoot Beach	1	1	West Bonita Beach Rd. from dune to East Tidal Barrier	3.5	12	T-WALL	2	2,750	12,028 CY/TON
			Length of East Tidal Barrier - 100' Miter Gate ; Barrier Along Bridge						
			Bonita Beach Rd near Barefoot Boat Club between East and Wast Tidal Barriers	9.5	12	T-WALL	2	1,750	
			Length of West Tidal Barrier - 100' Miter Gate ; Barrier Along Bridge						
			East Bonita Beach Rd. west of West Tidal Barrier	9.5	12	T-WALL	7	1,470	
	2		Beach Berm and Dune	3.5	12			8,110	12 FT dune, 75 ft berm Initial Construction: 749,800 CY Each Renourishment: 205,714 CY
Barefoot Beach Preserve	3	1	Beach Berm and Dune	3.5	12			7,475	12 FT dune, 75 ft berm Initial Construction: 41,400 CY Each Renourishment: 274,300 CY
Wiggins Pass State Park	4	1	Northern Jetty		12			430	Armor Rock- 23,100 TONS Bedding Rock- 3,280 TONS Filter cloth- 39,800 SF Geogrid - 39,800 SF
			Northern Barrier Wall Tie - In	-1.6	12	T-WALL	0	245	2,194 CY
			Wiggins Pass Tidal Barrier - 150'. Combo wall with Miter gates and Lift gate at navigation channel.						
			Southern Barrier Wall Tie - In	-3.3	12	T-WALL	0	270	
			Southern Jetty		12			575	Armor Rock- 22,700 TONS Bedding Rock- 3,830 TONS Filter cloth- 39,800 SF Geogrid- 39,800 SF
	5		Beach Berm and Dune	3.5	12			6,125	12 FT dune, 75 ft berm Initial Construction: 343,300 CY Each Renourishment: 398,271 CY
Vanderbilt	6	1	Beach Berm and Dune	3.5	12			6,900	12 FT dune, 75 ft berm Initial Construction: 378,100 CY Each Renourishment: 272,729 CY
Pelican Bay	7	2	NON STRUCTURAL ONLY						
Clam Pass State Park	8	2	NON STRUCTURAL ONLY						
Park Shore	9	3	West Seagate Dr from beach dune to Sluice Gates	2.5	14	T-WALL	0	1,050	13,711 CY
			(3x) Sluice Gate 19' x 24" (Inside Dia.)						
			East Seagate Dr.	6.6	14	T-WALL	1	3,450	
	10		Beach Berm and Dune	4.5/3.5	12			11,330	12 FT dune, 75 ft berm Initial Construction: 593,000 CY Each Renourishment: 300,457 CY
Naples	11	3	Northern Barrier Wall Tie - In	2.9	12	T-WALL	0	120	1,615 CY
			Doctors Pass Tidal Barrier -Length 100'. Miter gates at navigation channel.						
			Southern Barrier Wall Tie - In	1.65	12	T-WALL	0	125	
	12	5	East Tamiami Trail E	4	12	T-WALL	10	2,700	22,009 CY
	13		Tamiami Trail E East Tie-In	5.5	12	T-WALL	0	85	
			Length of East Tidal Barrier - 72'; Barrier Along Bridge						
			Naples Island East Barrier Tie-In	2	12	T-WALL	0	70	
	14		Naples Island	-2	12	T-WALL	1	935	
	15		Naples Island West Barrier Tie-In	-6	12	T-WALL	0	70	
			Length of West Tidal Barrier - 72'; Barrier Along Bridge						
			Tamiami Trail E West Tie-In	6.5	12	T-WALL	0	65	
	16		West Tamiami Trail E	6.8	12	T-WALL	39	9,925	
	17	3	Beach Berm and Dune	3.5	12			10,611	12 FT dune, 75 ft berm Initial Construction: 978,500 CY Each Renourishment: 546,071 CY
Marco Island	20	6	NON STRUCTURAL ONLY						

5.8. COST ESTIMATES

The baseline cost estimate for the proposed measures, tentative selected plan and the recommended plan were developed using MCACES in the Civil Works Work Breakdown Structure format. Quantities were calculated and provided by the designer engineers in the Norfolk District. Quantities are shown in Section 5.5.6 of this engineering appendix. The cost

estimate for each feature was escalated to the midpoint of construction using the most current indices for Civil Works Construction Cost Index System (CWCCIS) EM 1110-2-1304. For this project an Abbreviated Risk Analysis (ARA) and Cost and Schedule Riske (CSRA) were both performed on a 10% design. Since the design level is so low (10% design), this could inherently result in cost uncertainties that are captured by higher cost contingencies. For more information on the Cost Estimates and the Total Project Cost Summary (TPCS) and cost risk analysis performed on this project, refer to the Cost Engineering Sub-Appendix.

5.9. ENGINEERING RISK AND UNCERTAINITY

Risk is a measure of the probability (or likelihood) and consequences of uncertain future events. Risk analysis is a decision-making framework that explicitly evaluates the level of risk if no action is taken and recognizes the monetary and non-monetary costs and benefits of reducing risks when making decisions. A variety of variables and their associated uncertainties may be incorporated into the risk assessment of a coastal storm risk management study. Design conditions for major coastal and flood protection projects are often vague and design parameters contain large uncertainties.

One factor of uncertainty is the confidence of the water levels and waves as risk-based discussion involves the accuracy and appropriateness of the modeling performed for the FEMA SWFL Study, especially in shallow water conditions such as those for the back bay analysis. This risk and uncertainty extends to the FEMA SWFL Study storm suites used in both the Beach-*fx* and G2CRM models. ADCIRC, the current standard for hydrodynamic modeling, should not have problems providing accurate results in shallow water conditions. In shallow waters, some factors like bottom friction become more dominant relative to deeper waters, but for these studies ADCIRC must be validated across the study area, regardless of depth. For this study it is assumed that ADCIRC was properly validated for the FEMA study and that the mesh had adequate resolution within the study area. There is plenty of documentation provided on the FEMA SWFL Study and included in the references to support this assumption. Regarding the simulation of waves, unstructured SWAN uses the same mesh as ADCIRC so the high-resolution is preserved. Though the simulation of nearshore waves and waves in shallow water environments is complex, and can present greater uncertainty relative to storm surge simulations, from a practical standpoint there's no certainty that the alternatives would yield more accurate results. Moving to an offshore location would result in waves that are too large for a typical back bay area, producing unrealistic and biased economic results. It is possible to use a deeper water location and then transform back to shallow waters within the back bays, but SWAN coupled with ADCIRC, if properly validated, should perform better than linear wave theory or 1D models typically used for these nearshore transformations. An existing 2D models may have better representation of wave diffraction/refraction physics, but run into a myriad of issues, like being uncoupled from the surge, unrealistic flat bottom or static water level, and possibly inadequate representation of bottom friction, besides being computationally too expensive for large study areas. Thus, ADCIRC/SWAN should still be the preferred approach for water level and wave estimations in shallow water environments.

There is also some risk and uncertainty associated with the sea level rise rates incorporated in both the Beach-*fx* and G2CRM models as SLR rates are updated annually. This study utilized the 2018 rate (the rate at the start of the study) from the nearest NOAA tide gauge from which tide data can be evaluated: The Naples Pier gauge in Naples, Florida (NOAA Station 8725110) (rate was accessed in December 2019).

For calculating wave overtopping, the EurOtop method used both a deterministic and probabilistic approaches to analyze the wave overtopping for the design of the vertical wall and levee. To analyze risk of wave overtopping, different heights of the walls were analyzed and adjusted for different still water levels and waves based on the findings and results to meet protection for the 50 year life of the project. The approach to address this issue can be read in the Hydraulics, Hydrology and Coastal Engineering Sub-Appendix. When assessing a floodwall for risk analysis, the geotechnical engineer assumed two generalized, "worst-case" soil profiles to ensure safe wall performance. The structural engineer considered additional scour protection around the floodwall and if this will affect loads acting on the wall and realistic uplift loads. For more information on how risk was incorporated into the structural design of the floodwall, refer to the Geotechnical Engineering Sub-Appendix and the Structural Engineering Sub-Appendix for evaluations and hand calculations.

Risk associated with the cost is taken into consideration when the Abbreviated Risk Analysis (ARA) and Cost and Schedule Risk Analysis (CSRA) were performed on the project. Refer to the Cost Engineering Sub-Appendix for more detailed discussion on risk considered in the project cost.

5.10. CONSTRUCTABILITY

The primary constructability issues for the Collier County CSRM project are expected to be construction adjacent to existing structures and facilities, soft soils, and traffic impacts.

Many construction activities produce potentially damaging vibration levels, including pile driving and removal, concrete and asphalt demolition, soil compaction with a vibratory compactor, and excavation. Vibration amplitudes are typically measured and reported as peak particle velocity, usually in inches per second. The State of Florida does not have any criteria for maximum allowable vibration levels to prevent damage to adjacent structures. The criteria adopted for this project will be developed during the PED phase and will be based on criteria developed by others, such as other state Departments of Transportation and the Federal Transit Administration. For more information see Chapter 7 of the Geotechnical Sub-Appendix.

Construction adjacent to existing structures also means that the temporary construction right-of-way must be minimized. Construction in tight quarters tends to take longer, which increases costs, and may be more dangerous for the workers.

Construction near and along/across roadways will negatively impact traffic and may require temporary street or lane closures and traffic monitors. The floodwall alignments of the Collier County CSRM project cross numerous streets and parallel streets in some places. The

construction contractor will need an extensive traffic control plan and will have to perform extensive coordination with the City to provide public notice of traffic impacts.

As the project area is subject to tropical storms and hurricanes from May to November, and to windy and/or rainy weather during any time of the year, during construction, weather could also impact work and schedule. A schedule of monthly adverse weather delays is based upon NOAA Oceanic Atmospheric Administration (NOAA) or similar data for the project location will constitute the base line for monthly weather evaluation. A reasonable number of anticipated weather delay work days is obtained from the average number of days per month with precipitation greater than monthly average conditions. It should be a Contractor's responsibility to obtain information concerning weather conditions in the project area and a Contractor's schedule should include these anticipated days for delay.

5.11. RESILIENCY & ADAPTABILITY

Resiliency can be defined as the ability of the levee or structure to provide protection during events greater than the design event without total failure. Adaptability is defined as the quality of being able to adjust to new conditions or the capacity to be modified for a new use or purpose. The USACE Climate Change Adaptation Goal is to minimize impacts from climate change and maximize resiliency in the coastal landscape. USACE describes resilience as "the ability to anticipate, prepare for, respond to, and adapt to changing conditions and to withstand and recover rapidly from disruptions with minimal damage. The current 10% structural design of the Collier County CSRM project takes into consideration the effects of sea level rise, land subsidence and climate change as part of the design (i.e., heights of walls). For the analysis on SLR and when the heights of walls will need to be adjusted, please see the Hydraulics, Hydrology, and Coastal Engineering Sub-Appendix, which looks at the SLR increase from the start of the project to 100 years into the project life and for the discussion on water levels, confidence limits, and risk.

USACE Civil Works project designs should take into consideration how and if the design can adapt to the effects of sea level rise and climate change 100 years after the project is constructed and what adjustments can be made to the design to assure that the project can adapt and resiliency will still be possible with the project. For the Collier County CSRM, walls will be recommended to be built as T-walls. Due to the presence of bedrock close to the ground surface, sheet pile-based I-walls would not be feasible because sheet piles cannot be driven into bedrock. Surge barriers can also be raised to adapt to SLR. This has previously been done for a USACE Rock Island District project containing a sector gate.

During the PED Phase, the monitoring procedure for the project and adaptation will be written in the OMRR&R Manual (which will be discussed more in the next section). The OMRR&R manual will discuss in detail the specific thresholds for adaption, with lead times required for each action. Once constructed, the project will be placed in the USACE's Comprehensive Evaluation of Projects with Respect to Sea Level Change (CESL) tool to provide additional forecast for potential adaptation. The purpose of this tool is to inventory and assess the

vulnerability of existing USACE projects to the effects of RSLR and provide added benefits to other USACE activities. CESL tool is also used to:

1) Provide assistance in evaluating robustness of projects to potential SLR.
2) Screen projects to develop a prioritization process that identifies those that need to take SLR into consideration immediately, and those that can take longer. This screening process will build on existing tools, including NOAA tide gauge and sea-level visualizations, the USGS Coastal Vulnerability Index, and the data in CorpsMap. The prioritization strategy will encompass the entire portfolio of projects over the whole life cycle and be used to guide the more detailed project-scale evaluation.
3) During screening, USACE with the local sponsor will identify and capture (some of) the information about each project that is needed to perform the more detailed project-scale evaluation.
4) Assist the districts in performing a more detailed project-scale evaluations according to the prioritization process. This phase will identify current SLR scenarios, identify triggers or thresholds at which future measures should be considered, and provide measures appropriate to support sustainable performance under potential SLC scenarios.
5) Provide district and division commanders the ability to communicate with other federal and state agencies as well as local sponsors what are multiple climate change impacts on their areas, projects, etc.

5.12. MONITORING & INSPECTION

Once the project has been constructed and turned over, USACE will provide an operations, maintenance, repair, replacement, and rehabilitation (OMRR&R) manual which will be written specifically for the local sponsor, the Collier County, who will have the primary responsibility for operating and maintaining the project. The intent of the document is to provide the local sponsor with some clear and comprehensive guidance on the operation and maintenance of beach dunes, floodwalls, and other flood control structures. It will describe how to plan and prepare for high water and storm events, and lays out steps to take during emergencies that will help reduce the threat of flooding. The manual will also explain the types of assistance that the U.S. Army Corps of Engineers can provide to a community before, during, and after a flood. Monitoring and inspections must occur to ensure that the project functions as designed and that the local sponsor confirms to all OMRR&R recommendations and requirements that will assist in functionality of the project. USACE will inspect the project each year with Collier County (the local sponsor). USACE conducts two types of dune/floodwall inspections: Routine Inspection and Periodic Inspection. Routine Inspection is a visual inspection to verify and rate dune/floodwall system operation and maintenance. It is typically conducted each year for all dune/floodwalls in the USACE Safety Program. Periodic Inspection is a comprehensive inspection conducted by a USACE multidisciplinary team that includes the local sponsor and is led by a professional

engineer. USACE typically conducts this inspection every five years on the federally authorized levees in the USACE Safety Program. Periodic Inspections include three key steps:

1) Data collection - A review of existing data on operation and maintenance, previous inspections, emergency action plans and flood fighting records;
2) Field inspection - Similar to the visual inspection for a Routine Inspection, but with additional features;
3) Final report development - A report including the data collected, field inspection findings, an evaluation of any changes in design criteria from the time the levee was constructed, and additional recommendations as warranted, such as areas that need further evaluation.

Both Routine and Periodic Inspections result in a final inspection rating for operation and maintenance. The rating is based on the floodwall inspection checklist, which includes 125 specific items dealing with the operation and maintenance of beach dunes, floodwalls, interior drainage, pump stations, and channels. Each dune/floodwall segment receives an overall segment inspection rating of Acceptable, Minimally Acceptable, or Unacceptable. If a floodwall system comprises one or more segments of the project then the overall project system rating is the lowest of the segment ratings. The local sponsor must maintain the dunes and floodwall to at least the minimally acceptable standard to remain eligible for federal rehabilitation assistance through the USACE Rehabilitation and Inspection Program (PL 84-99).
USACE also shares the results with FEMA, to help inform decisions about levee accreditation for flood insurance purposes. The inspection ratings are available in the National Levee Database.

The local sponsor should be prepared to carry out maintenance activities on all flood control structures every year. Regular maintenance is critical, because many types of problems will escalate exponentially when left unchecked. There are many ongoing requirements of which one should be aware. For example, debris and unwanted growth need to be removed from beach dunes, riprap, and the areas adjacent to floodwalls, and from channels and waterways. An animal control program is needed for the levees, and any burrows that are found need to be filled properly. Vegetation control on beach dunes are extremely important as vegetation stabilizes the sand on the dunes. Vegetation on the dunes shall be replanted as needed to maintain the dunes. Metal gates and other components need to be painted and greased periodically. Concrete damage needs to be identified and repaired early or it will get worse, especially in northern climates where freeze-thaw damage becomes a factor. Beyond these examples of ongoing maintenance, there are also more significant repairs that will be necessary from time to time. On occasion, the local sponsor may have to add stone to control an erosion problem, or do some major earthwork to repair an embankment. Metal culverts running through a levee will have to be completely replaced from time to time, because they typically don't last more than about fifty years. Pump stations also need to be completely overhauled periodically. Routine maintenance is expected in any project and can be planned for in advanced.

To assist with monitoring, certain tools and instruments are needed and measurements are required. Gauges are recommended to be installed in all areas where storm surge barriers are being constructed. This is to assist in providing accurate real-time readings of the water surface

elevations in each area. Other Geotechnical instruments are needed to measure movement of the structures and periodic surveys are required to monitor for possible settlement.

CHAPTER 6 PRECONSTRUCTION ENGINEERING AND DESIGN (PED) CONSIDERATIONS

6.1. GENERAL

During PED, design refinements will be conducted for all plan structural elements based on new field investigations and analyses. This chapter will discuss, not only what information and field investigation will be needed to achieve a final design, but also, how and what has been proposed in this study may be changed or adjusted.

6.2. UPDATED SURVEYS

It is recommended that a topographic survey be performed during PED in areas of the County structures are proposed. New surveys may require an adjustment in the proposed height, length, and alignment of all the structural elements in the TSP. It is also recommended that updated hydrographic surveys be collected in the waterways where surge barriers are being proposed, since shoaling may have occurred in several of the waterways and more accurate measurements of the waterway depth should be taken. A more recent and comprehensive topographic and hydrographic survey will be required in order to develop plans and specifications.

6.3. BEACH BERM AND DUNE DESIGN REFINEMENT

During the PED phase, the beach alignment will be refined at each R-Monument using new surveys collected along shore. This will provide for a refined project baseline to be used during nourishment construction as well as a refined beach design that optimizes the placement of the beach dune and berm at each R-monument. The beach berm widths may need to be modified depending on volume requirements in each reach. Beach berm and dune design refinement will be necessary during the PED phase as well as prior to each subsequent renourishment during the project's 50 year life cycle.

During PED, potential impacts to hardbottom with the TSP design beach should be reevaluated and refined based on the optimized beach placement of the dune and berm.

6.4. SAND SOURCE INVESTIGATION

A major assumption for this project is that sand will be available from in the offshore BOEM sand source historically utilized and leased by Collier County. During PED, a geotechnical investigation may be required to confirm the quality of the sand in the proposed borrow sites which are not currently used by the County and the quantity of beach quality sand available. Areas near the proposed borrow sites may need to also be investigated for the potential expansion of the borrow site if necessary in the future.

6.5. FLOODWALL DESIGN REFINEMENT

During the PED phase, subsurface explorations will be conducted along floodwall alignments to supplement the existing information. Information from all subsurface explorations will be used to develop site-specific subsurface cross sections of refined computer design of I-Walls and axial and lateral load capacity, settlement, and footing uplift pressure for T-Walls. The L-PILE computer program will be used to determine lateral pile capacity and deflections. The results of these analyses could result in changes to the assumed embedment depth of the piles (shorter or longer).

Additionally design, alignment, and selection of the types of floodwalls and road closures should be optimized per detailed hydrodynamic modeling of each of the proposed areas so as to ensure the construction of the floodwalls do not induce flooding in the surrounding areas and minimally impact existing hydrodynamics.

6.6. ALIGNMENT AND EASEMENTS

During the PED Phase, more information and data will be collected, including real estate information. The collection of new information and data, may require adjustments to the proposed alignments, if easements cannot be acquired in certain areas. Real estate requirements for the study area consist of Flowage Easements (FE), Flood Protection Levee Easements (FPLE), Temporary Work Area Easement (TWAE), and Bank Protection Easement (BPE). These easements will be needed to provide adequate construction room to build proposed flood risk management features and secure lands needed for Operations and Maintenance (O&M). More information on easements and real estate requirements can be read in the Real Estate Appendix in this study report.

6.7. STORM SURGE BARRIERS (TIDE GATES) REFINEMENT

During the PED phase, subsurface explorations will be conducted along the storm surge barrier alignments to supplement the existing information. Information from all subsurface explorations will be used to develop site-specific subsurface cross sections for refined design of these features, including axial and lateral pile capacity and settlement. At locations where soft clay layers exist, the effect of down drag will be evaluated. The L-PILE computer program will be used to determine lateral pile capacity and deflections. The results of these analyses could result in changes to the assumed embedment depth of the piles (shorter or longer) and/or assumed number of piles. Sizes, depths of sills and final opening sizes will be based upon accurate topographic and detailed field investigation. The operator systems that are employed will be based upon available room and power sources. Analysis of these structures shall be conducted using commercial off the shelf software such as Rapid Interactive Structural Analysis (RISA) and hand calculations as applicable. To the greatest extent possible, we will be employing the use of American Institute Steel Construction steel sections to design and construct these items.

Additionally design, alignment, and selection of the types of surge barriers should be

optimized per detailed hydrodynamic modeling of each of the proposed barrier areas so as to ensure the construction of the barriers do not induce flooding in the surrounding areas and minimally impact existing hydrodynamics.

6.8. INTERIOR FLOODING ANALYSIS REFINEMENT

During the PED phase, H&H will refine the interior drainage analysis to more accurately provide choices of interior drainage relief. The analysis would entail the use of the HEC-HMS (Hydrologic Engineering Center – Hydrologic Modelling System) software version 4.3 or the latest model available with the guidance of Engineering Manual 1110-2-1413. Various rainfall frequencies from the 2 year to 500 year 24 hour point rainfall from the NOAA Atlas 14 will be used as the input. The refined interior flooding analysis shall focus on optimizing the number of pumps and pump stations required for each planning area with a surge barrier and potential issues with induced flooding on the unprotected side of all floodwalls and barriers, especially those with large pump stations discharging onto the unprotected side.

CHAPTER 7 REFERENCES

The following are references that are applicable to this main engineering appendix. References for the sub-appendix shall be mentioned in the sub-appendices.

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