ENGINEERING APPENDIX

MIAMI-DADE COUNTY BACK BAY COASTAL STORM RISK MANAGEMENT FEASIBILITY STUDY

MIAMI-DADE COUNTY, FLORIDA

APPENDIX B

April 2020

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U.S. Army Corps
of Engineers
Norfolk District

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TABLE OF CONTENTS

CHAPTER 1 INTRODUCTION 1

1.1. DESCRIPTION OF PROJECT AREA AND VICINITY 1

1.2. SCOPE OF ENGINEERING APPENDIX AND ENGINEERING ANALYSIS 5

CHAPTER 2 EXISTING INFORMATION AND DATA 6

2.1. SURVEY DATA 6

2.2. SOIL BORINGS AND GEOTECHNICAL REPORTS 6

2.3. HYDROLOGY AND HYDRAULICS DATA 6

2.4. NOAA VACA KEY TIDAL GAGE RECORD 7

2.5. PREVIOUS STUDIES AND REPORTS 7

CHAPTER 3 PHYSICAL CONDITIONS 8

3.1. CLIMATE 8

3.2. HORIZONTAL AND VERTICAL DATUMS 8

3.3. WINDS & WAVES 9

3.4. ASTRONOMICAL TIDES & WATER LEVELS 16

3.4.1. ASTRONOMICAL TIDES 16

3.4.2. WATER LEVELS 20

3.5. STORMS 21

3.5.1. TROPICAL STORMS 21

3.5.2. HURRICANES 22

3.5.3. HISTORICAL STORMS 22

3.6. SEA LEVEL RISE 29

3.6.1. RELATIVE SEA LEVEL RISE (RSLR) 29

3.6.2. LAND SUBSIDENCE 31

CHAPTER 4 FEMA SOUTH FLORIDA STORM SURGE STUDY (SFLSSS) 32

4.1. PURPOSE OF SFLSSS STUDY 32

4.2. STORMS AND WATER 32

CHAPTER 5 ENGINEERING EVALUTION 35

5.1. GENERAL 35

5.2. WATER LEVELS 35

5.3. PROJECT ALIGNMENT 37

5.4. GENERATION II COASTAL RISK MODEL (G2CRM) 44

5.4.1. METEROLOGICAL DRIVING FORCES 44

5.4.2. PROTECTIVE SYSTEM ELEMENTS 46

5.5. FLOODWALL 46

5.6. GATE CLOSURES 49

5.7. STORM SURGE BARRIERS, TIDE GATES, AND FLAP GATES 55

5.7.1. SECTOR GATES 55

5.7.2. MITER GATES 56

5.7.3. SLUICE GATES 57

5.7.4. FLAP GATES 58

5.8. INTERIOR FLOODING ANALYSIS 59

5.9. WAVE FORCES ON VERTICAL WALL 60

5.10. WAVE OVERTOPPING 60

5.11. QUANTITY ESTIMATES 61

5.12. COST ESTIMATES 62

5.13. ENGINEERING RISK AND UNCERTAINITY 62

5.14. CONSTRUCTABILITY 63

5.15. MONITORING & INSPECTION 65

CHAPTER 6 PRECONSTRUCTION ENGINEERING AND DESIGN (PED) CONSIDERATIONS 68

6.1. GENERAL 68

6.2. FLOODWALL DESIGN REFINEMENT 68

6.3. UPDATED SURVEYS 68

6.4. ALIGNMENT AND EASEMENTS 68

6.5. STORM SURGE BARRIERS (TIDE GATES) REFINEMENT 69

6.6. INTERIOR FLOODING ANALYSIS REFINEMENT 69

CHAPTER 7 REFERENCES 70

SUB APPENDICES 71

1. STRUCTURAL SUB APPENDIX 71

2. GEOTECHNICAL SUB APPENDIX 71

3. HYDROLOGY, HYDRAULICS, & COASTAL SUB APPENDIX 71

4. COST ENGINEERING SUB APPENDIX 71

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LIST OF ACRONYMS AND ABBREVIATIONS

ADCIRC	ADvanced CIRCulation Model
AEP	Annual Exceedance Probability
CSRM	Coastal Storm Risk Management
DEM	Digital Elevation Model
ERDC	USACE Engineering Research and Development Center
FEMA	Federal Emergency Management Agency
FWP	Future With Project
FWOP	Future Without Project
ft.	feet
HEC-HMS	Hydrologic Engineering Center-Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center-River Analysis System
H&H	Hydrology and Hydraulics
HH&C	Hydrology, Hydraulics, & Coastal
HWM	high water mark
LIDAR	Light Detection and Ranging
MSL	mean sea level
NAVD 88	North American Vertical Datum of 1988
NGVD 29	National Geodetic Vertical Datum of 1929
NOAA	National Oceanic and Atmospheric Administration
PDT	Project Delivery Team
RSLR	Relative Sea Level Rise
SACS	South Atlantic Coastal Study
SFL	South Florida
SFLSSS	South Florida Storm Surge Study
SFWMD	South Florida Water Management District
SLR	sea level rise
SWAN	Simulating WAves Nearshore
SWL	Stillwater level
TSP	Tentatively Selected Plan
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey

CHAPTER 1 INTRODUCTION

1.1. DESCRIPTION OF PROJECT AREA AND VICINITY

The Miami-Dade County Back Bay Coastal Storm Risk Management (CSRM) Study is a study conducted by the U.S. Army Corps of Engineers, Norfolk District and the Local Sponsor, Miami-Dade County, to reduce Miami-Dade County's risk of coastal storm damages and impacts to the year 2079. Miami-Dade County is located on the east coast, which is located in the Southeastern portion of the State of Florida (**Figure 1.1**).

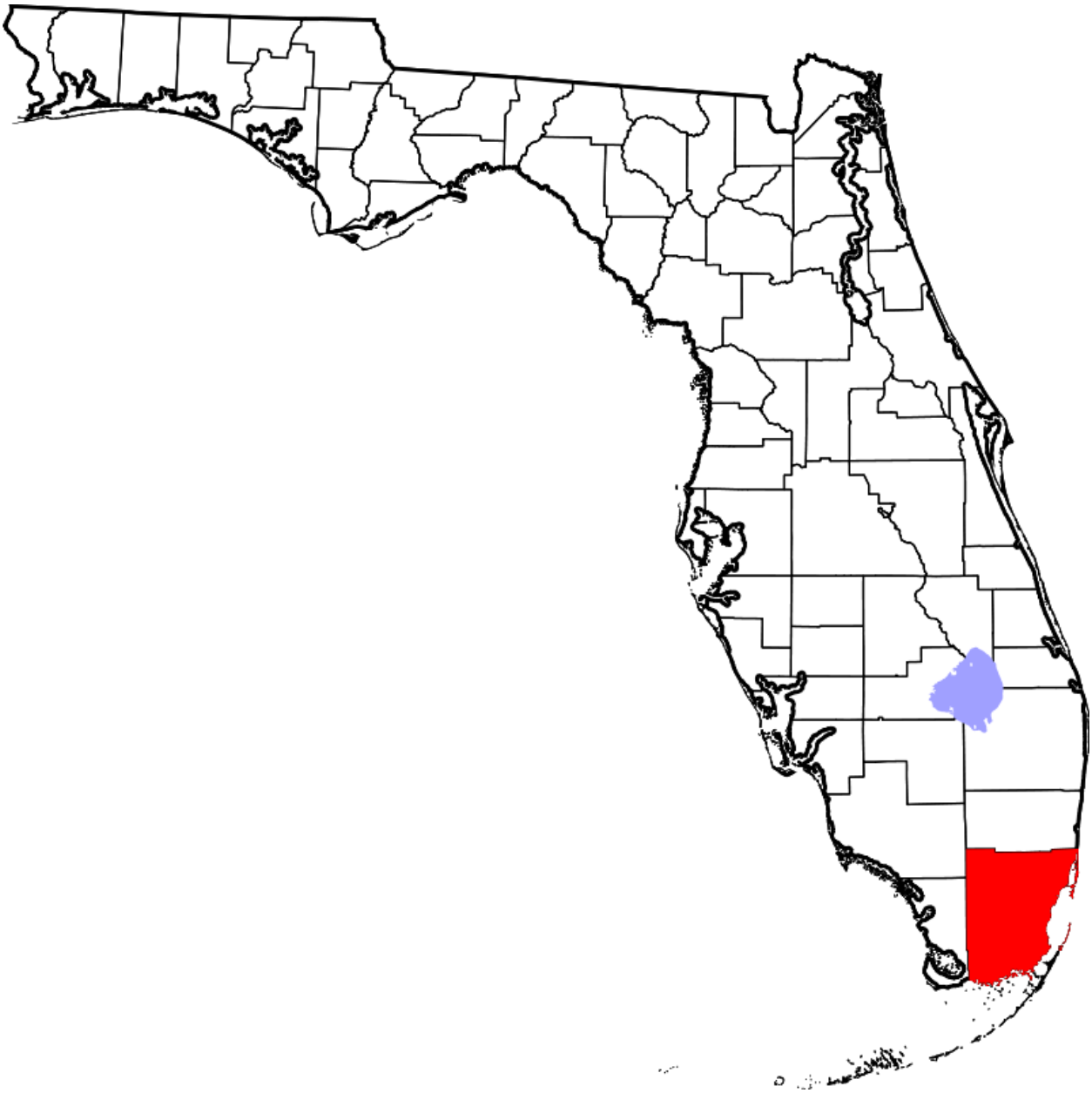


Figure 1.1. Location Map of Miami-Dade County (in red) within the State of Florida.

The County is home to 34 incorporated cities and is the third largest county in Florida with one third of the County located in the Everglades National Park. The highly urbanized metropolitan area is made up of City of Miami, City of Miami Beach, and 32 other municipalities in Miami-Dade (**Figure 1.2**) Miami has a high natural elevation of 22.75 feet NAVD88 (USGS SIR 2014-5162 Vers 1.2, July 2016) along the Miami Rock Ridge in Coconut Grove. The average elevation is only about 4.86 feet NAVD88 (USGS SIR 2014-5162 Vers 1.2, July 2016). There is also an Atlantic Coastal Ridge immediately west of the Biscayne Bay that has an elevation as high as 24.89 feet NAVD88 (USGS SIR 2014-5162 Vers 1.2, July 2016). The Biscayne Bay is west of the City of Miami Beach and is divided from the Atlantic Ocean by the many barrier isles along the coast. Miami-Dade County also includes municipalities on barrier islands which are low-lying.

Miami-Dade County is at the eastern edge of the Florida Platform, which is a carbonate plateau created millions of years ago. Eastern Dade is composed of Oolite limestone (grainstone) while western Dade is composed mostly of Bryozoa limestone (poorly to well indurated, sandy, fossiliferous limestone (grainstone and packstone)). Miami-Dade is among the last areas of Florida to be created and populated with fauna and flora, mostly in the last glacial period.

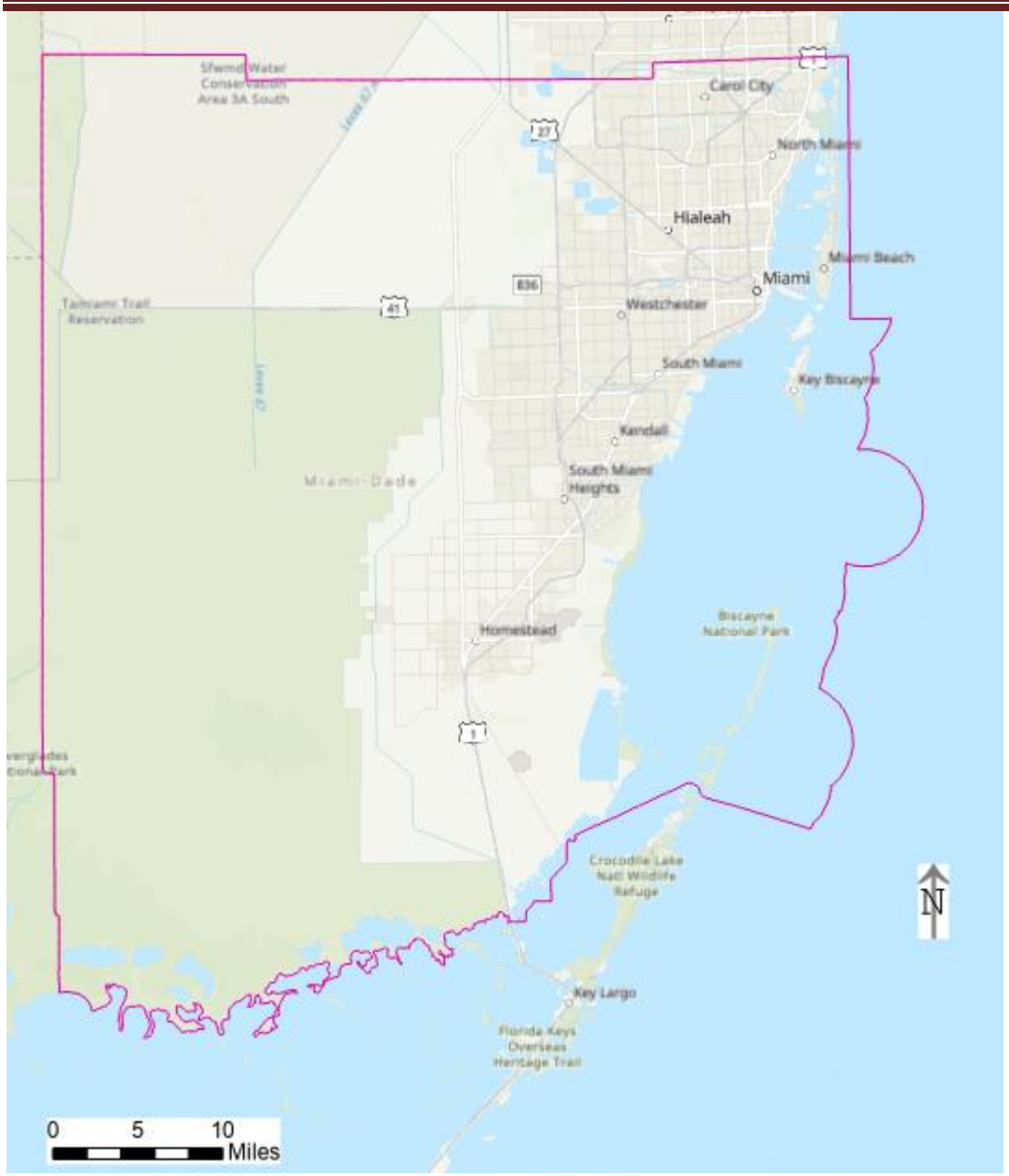


Figure 1.2. Map image of Miami-Dade County.

The low elevations and connections to the Biscayne Bay place a significant percentage of the Miami-Dade area at risk of coastal and tidal flooding from high tides, hurricanes, and tropical storms. The U.S. Geological Survey (USGS) Report, "National Assessment of Coastal Vulnerability to Sea Level Rise," identifies the Miami-Dade coast including the Biscayne Bay as an area of high vulnerability to sea level rise. At a national scale, the Coastal vulnerability index for the Atlantic Coast provides a preliminary overview of the relative susceptibility of the Nation's coast to sea- level rise. Geomorphology, regional coastal slope, tide range, wave height,

relative sea-level rise and shoreline erosion and accretion rates all contribute to the Coastal Vulnerability Index.

The memorable hurricanes are the Great Miami Hurricane in 1926, Betsy in 1965, Andrew in 1992, Irene in 1999, Hurricanes Katrina and Wilma in 2005, and Hurricane Irma in 2017. The study has four major focus areas (**Figure 1.3**) for proposed structural solutions, which is the main focus of this Engineering Appendix. Each area is based on the sub watersheds within Miami-Dade County.

Refer to the Main Report for discussion of the nonstructural solutions and the formulation of all focus areas considered.

Update: This Appendix discusses the structural solutions considered for alternative 7 mentioned in the main report. The current TSP is alternative 8 and does not include the Edgewater floodwall alignment. Any discussions about the Edgewater alignment are to be ignored. A revision of the Engineering Main Appendix and the Sub Appendices will be provided before the Final Report.

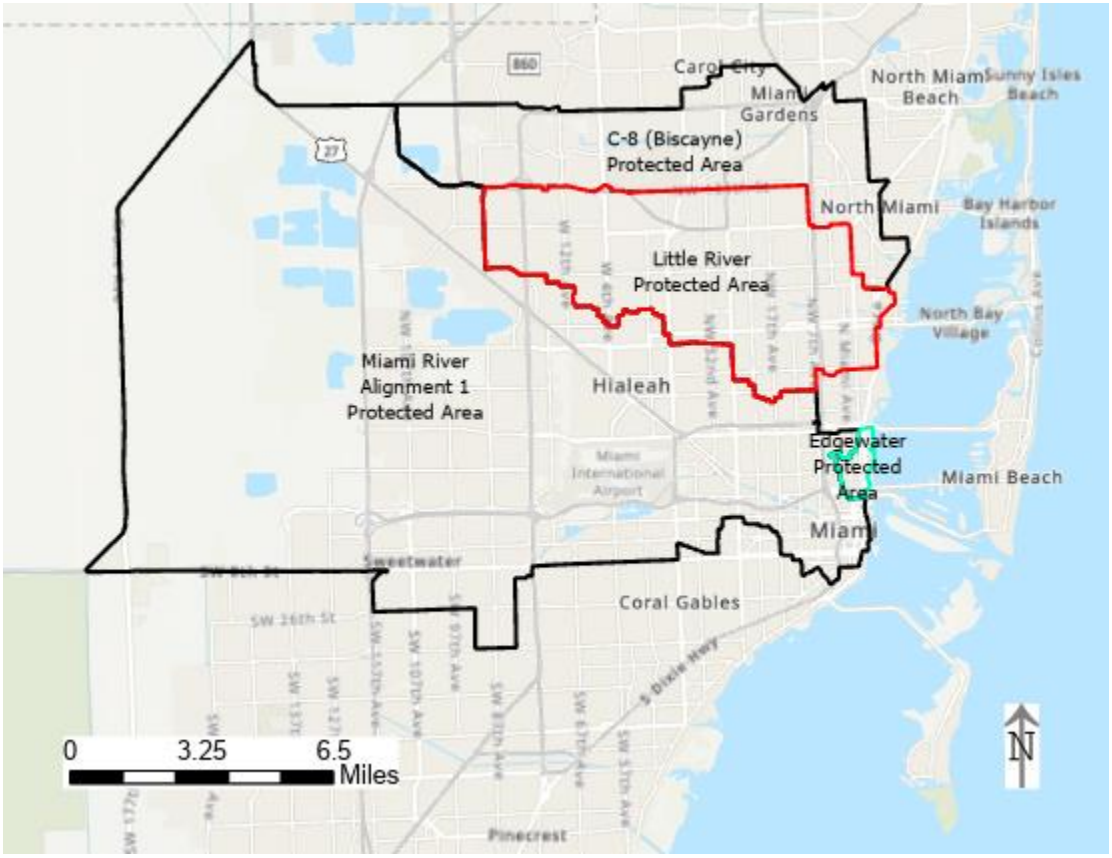


Figure 1.3. Focus Areas of the Miami-Dade County Back Bay CSRM.

The Focus Areas for the proposed structural solutions were determined by watershed boundaries and are as follows:

- Miami River Watershed
- Edgewater Watershed (now deleted)
- Little River Watershed
- Biscayne Canal Watershed

Note: There may be a structural measure proposed on Arch Creek, the Coral Gables Waterway, and on the canal near the Snapper Creek Lakes Club. More information is located in the HH&C Sub Appendix. The full analysis and quantities associated with the latest proposed alignment of protection will be updated for this Draft Appendix before the Final Report.

1.2. SCOPE OF ENGINEERING APPENDIX AND ENGINEERING ANALYSIS

The Miami-Dade County Back Bay CSRM Study will address potential structural and non-structural solutions in terms of mitigating the impacts of flooding. This Engineering Appendix discusses the preliminary engineering and 10% design work conducted of the structural elements and measures of the Miami-Dade County Back Bay CSRM Study. The preliminary design includes the evaluation of existing information and data, the evaluation of floodwalls, review and use of water levels and wave data from the existing Federal Emergency Management Agency (FEMA) South Florida Storm Surge Study (SFLSSS), estimation 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, and Hydrology, Hydraulics, & Coastal Engineering (HH&C).

CHAPTER 2 EXISTING INFORMATION AND DATA

The first goal of this study was to analyze and review any existing data or information that would assist with the analysis of the 10% level of design. For the Miami-Dade County Back Bay CSRM Study, 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

For this study, topographic and bathymetric survey data were available for all the specific focus areas through the FEMA SFLSSS. The SFLSSS used LiDAR and bathymetry for the basis of the elevation grid. 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). For this study the DEM included topographic and bathymetric data and was used in the Hydrology and Hydraulics (H&H) analysis. The data was also used to analyze existing ground surface elevations and determining the heights of wall structures, closures, and surge barriers. The DEM modeling grid from the FEMA SFLSSS is described in more detail in the HH&C Sub Appendix.

2.2. SOIL BORINGS AND GEOTECHNICAL REPORTS

Existing soil borings and geotechnical reports located near the Miami-Dade County Back Bay CSRM Study proposed project features were evaluated to determine general subsurface soil conditions. A summary of the information obtained is given in this section and a more detailed discussion of geotechnical conditions is given in the Geotechnical Sub Appendix.

2.3. HYDROLOGY AND HYDRAULICS DATA

USACE Engineer Regulation 1165-2-21 states “Water damage associated with inadequate carrying capacity of man-made structures should be designated as a flood problem or a local drainage problem in a manner consistent with the structure's classification as flood damage

reduction works or a part of a storm sewer system. Man-made structures that convey sanitary sewage or storm runoff, or a combination of sanitary and storm sewage, to a treatment facility will not be classified as flood damage reduction works". It is the local sponsor's responsibility to upgrade the storage capacity of a storm water system that cause flooding. However, the project alignment associated with construction of a floodwall can prevent runoff (from rainfall) and storm water from draining out of the interior of the project alignment. Several Draft report studies were provided by Miami-Dade County. The Draft Reports included assessments of some watersheds and recommended the sizing of forward pumps needed to handle certain frequency events. The South Florida Water Management District (SFWMD) operates and is responsible for the water control structures on the canals. More discussion of the interior flooding analysis is included later in this appendix introduction and in the HH&C Sub Appendix.

2.4. NOAA VACA KEY TIDAL GAGE RECORD

The records for the National Oceanic and Atmospheric Administration (NOAA) tidal gauge at Vaca Key extend from 1970 until the present and are considered representative of the sea level rise history at and near Miami-Dade County. In the HH&C Sub Appendix Attachment 2 is a white paper prepared by the USACE Jacksonville District that discusses why the Vaca Key tide gauge was used for adding the SLR to SWL elevations. Further discussion on the tide gauges and the information it provides can be read in **Chapter 3**.

2.5. PREVIOUS STUDIES AND REPORTS

Miami-Dade County has previously conducted several engineering analyses. The reports from these studies were reviewed to better understand the flooding issues of the County in the four focus areas and to decide what information and data from these studies could be used in the Miami-Dade County Back Bay CSRM Study. The previous study reports provided by the Miami-Dade County are mentioned in the main report.

CHAPTER 3 PHYSICAL CONDITIONS

3.1. CLIMATE

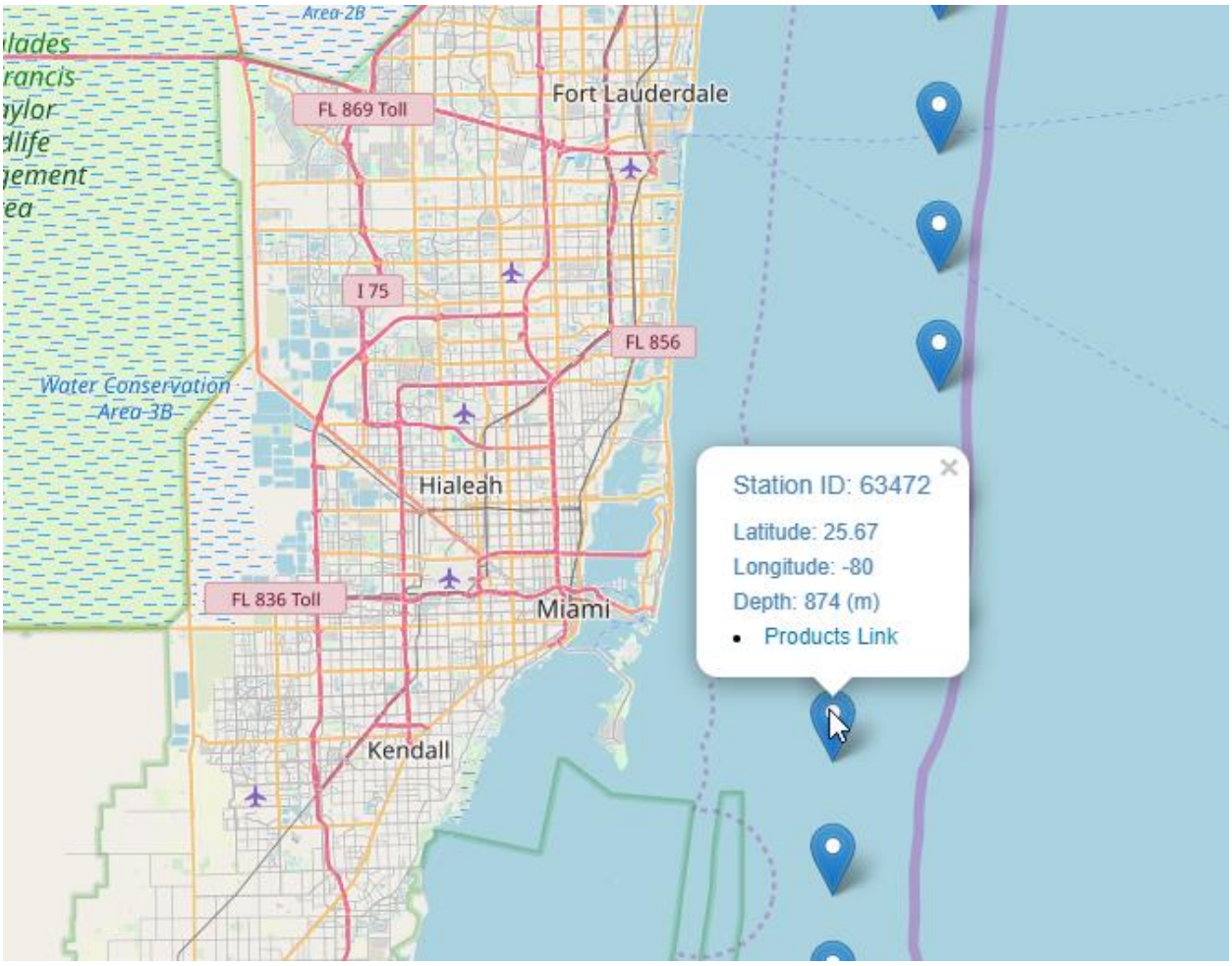
Miami-Dade's climate is tropical monsoon located in a tradewind coast with the mean monthly temperatures above 64 degrees Fahrenheit follow by a dry season usually occurring at or soon after the "winter" solstice. A tropical monsoon climate is controlled by the monsoon circulation and has a seasonal change in wind direction. Winters feature an offshore air flow (air moving from land toward water). The change in wind direction is due to the difference in the way water and land heat. In the summer the winds are from the water and induce a local sea breeze circulation that produces frequent thunderstorms. The summers are hot and humid and the winters are short and warm. Miami receives ample rainfall and is one of the highest among major cities in the United States. Miami has an average annual rainfall of 61 inches. Most of this rainfall occurs from mid-May through early October with average monthly amounts of 6 inches from May to October. Flooding in Miami also occurs from the combined effects of heavy precipitation and tidal events. The largest tidal events, which are unrelated to storm or weather conditions, are the King Tides which occur multiple times a year in the spring and fall seasons. According to the Miami-Dade County information on King Tides, they are the highest predicted high tides of the year and cause sunny day flooding. There are gravitational pulls on Earth; when the Moon (Perigee) is closest to the Earth and when the Sun (Perihelion) is closest to the Earth. When the Sun, Moon, and Earth align, solar and lunar gravity creates very high tides known as King Tides. The Florida King Tides occur annually between September and November in Miami because the seasonal winds drive the warmer water levels higher during the fall and impacts have been made worse by recent and continued sea level rise. Hurricanes and tropical storms can also have high tides and heavy rainfall.

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 U.S. 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 wind and wave data from the WIS discussed in this chapter was used to illustrate the prevailing wind and wave directions only. The wave data used in the overtopping analysis for the floodwalls came from the FEMA SFLSSS. The wind and wave directions near the study area are illustrated at two WIS stations, one closer to shore in deep water and another in deeper water in **Figure 3.1.** A shallow water WIS station was not located near the Miami-Dade focus area. Hindcast data was available for each of these stations from the years 1980 – 2014.



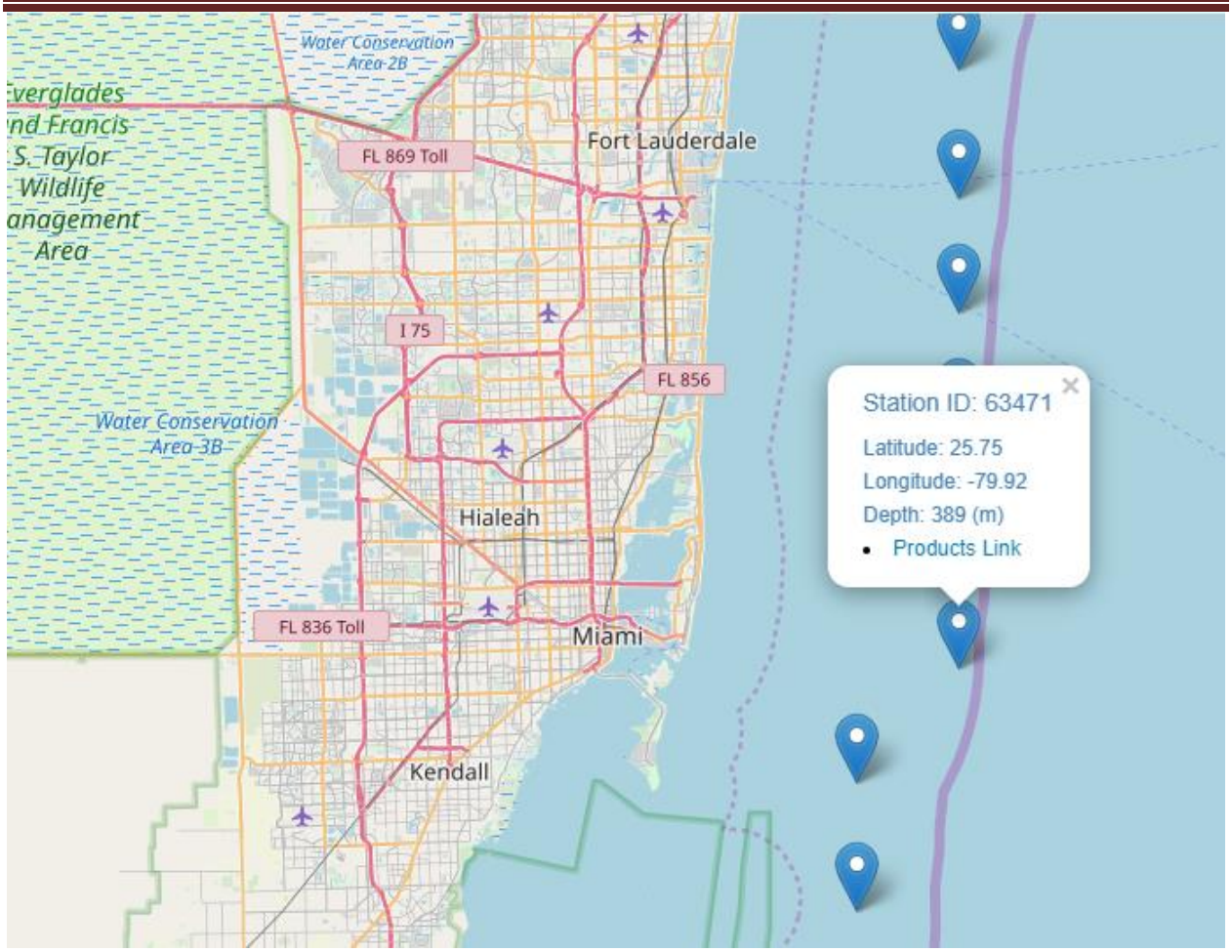


Figure 3.1. WIS Stations locations: 63472 (deeper) and 63471 (deep).

For both stations, the prevailing wave directions impacting Miami-Dade County are those from northeast and southeast. 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 show plots of wave roses and summaries of wave conditions off the coast of the City of Miami for deeper water and deep water conditions respectively.

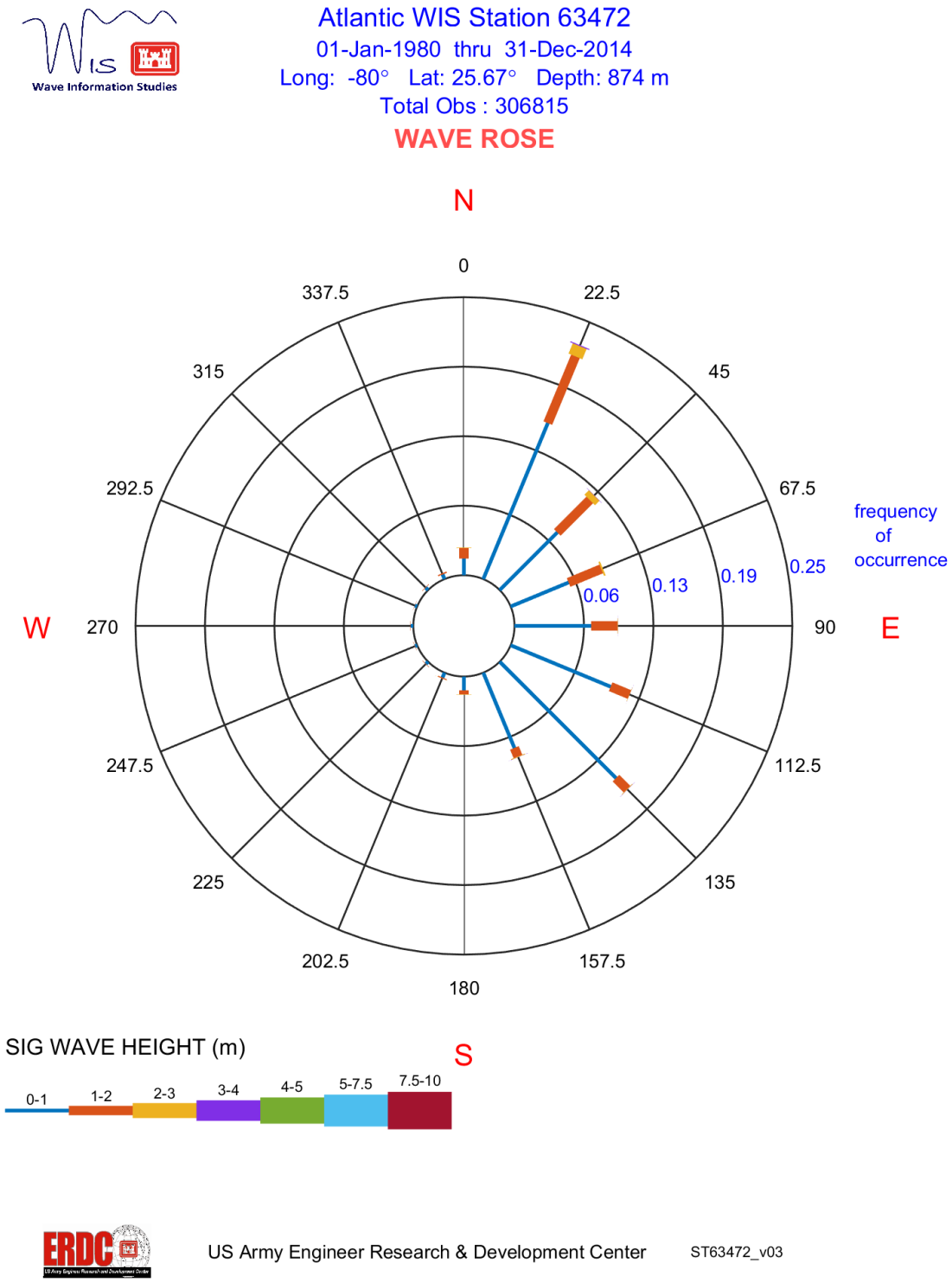


Figure 3.2. Deeper Water Wave Rose (from WIS Data).

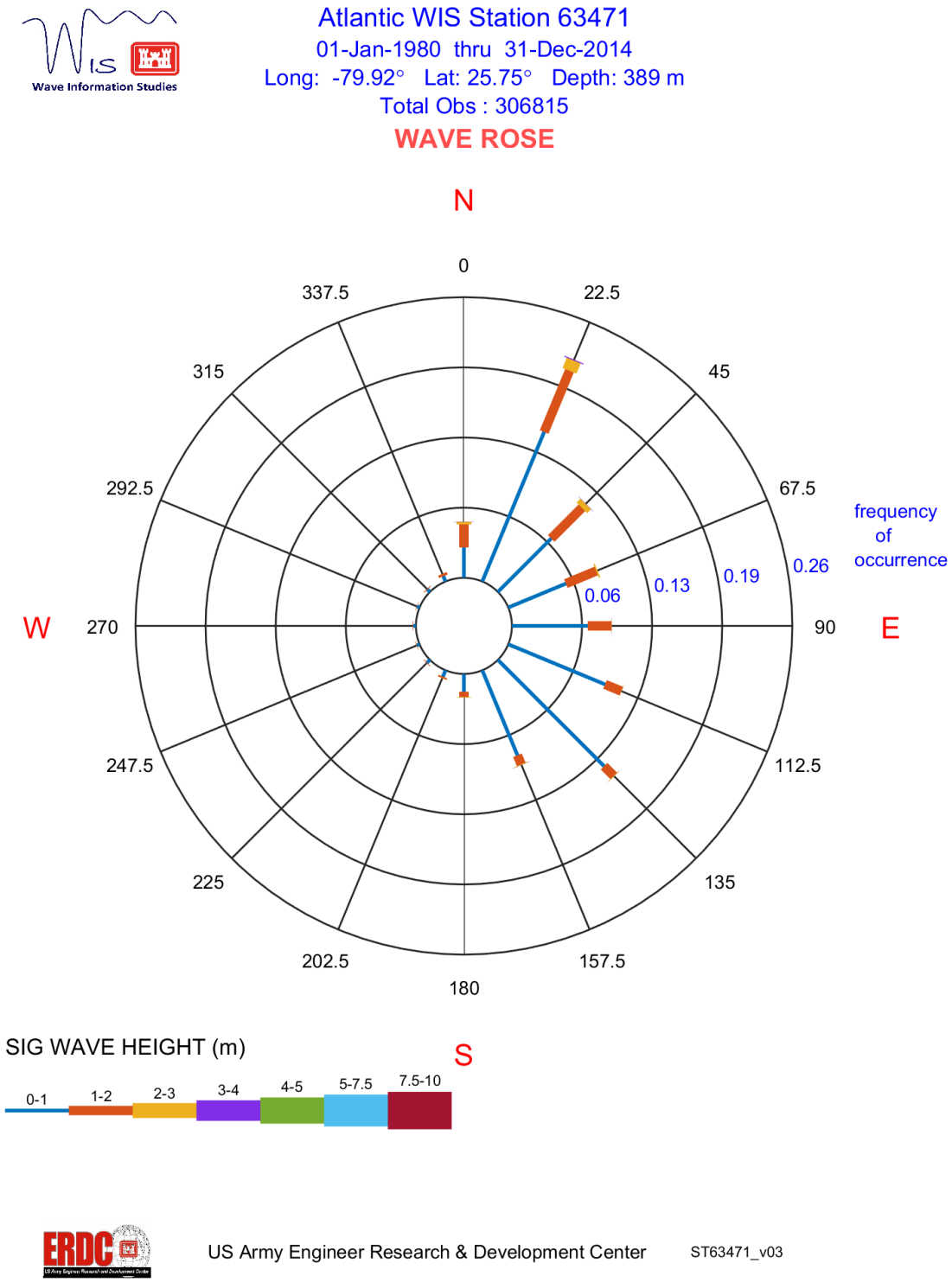


Figure 3.3. Deep Water Wave Rose (from WIS Data).

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

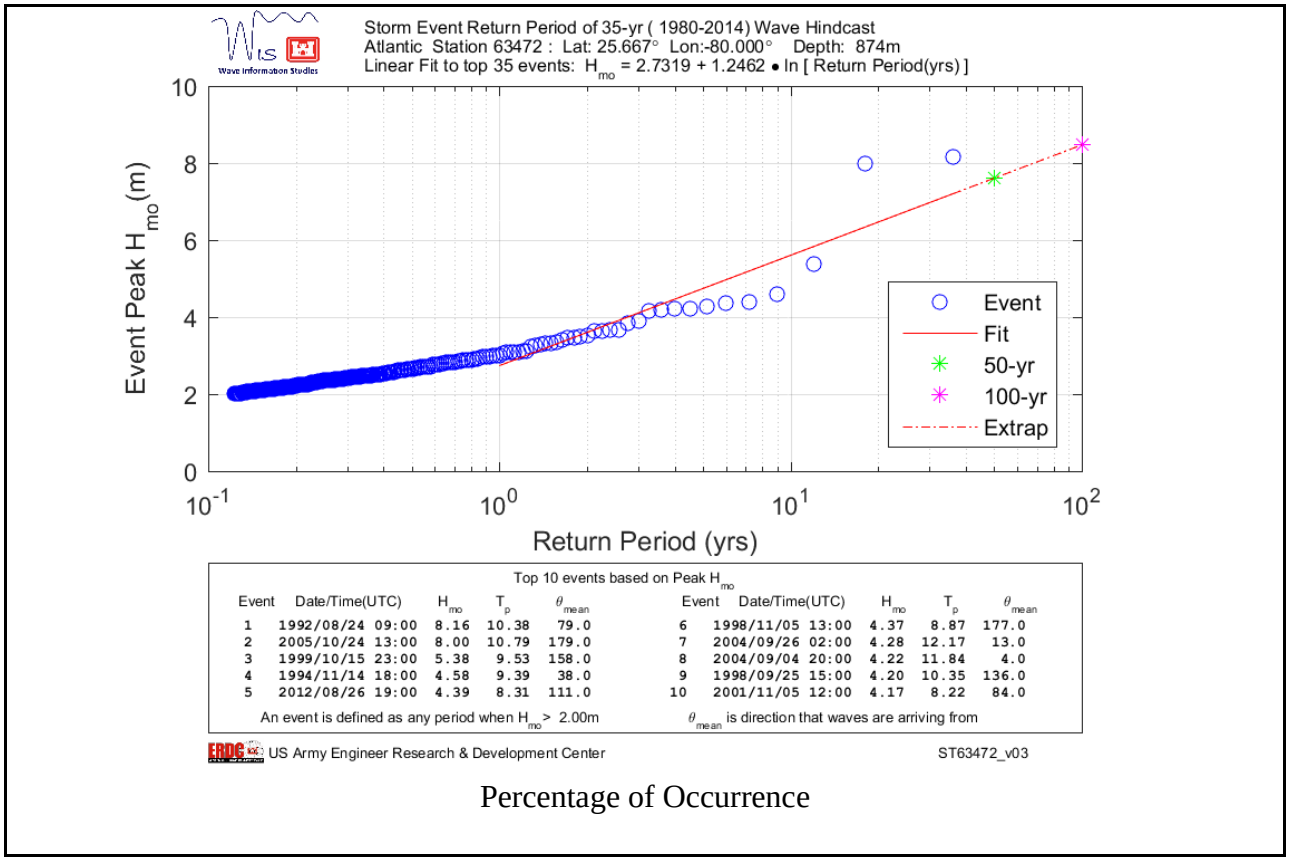


Figure 3.4. Image of Deeper (Station 63472) water storm event return period (from WIS data).

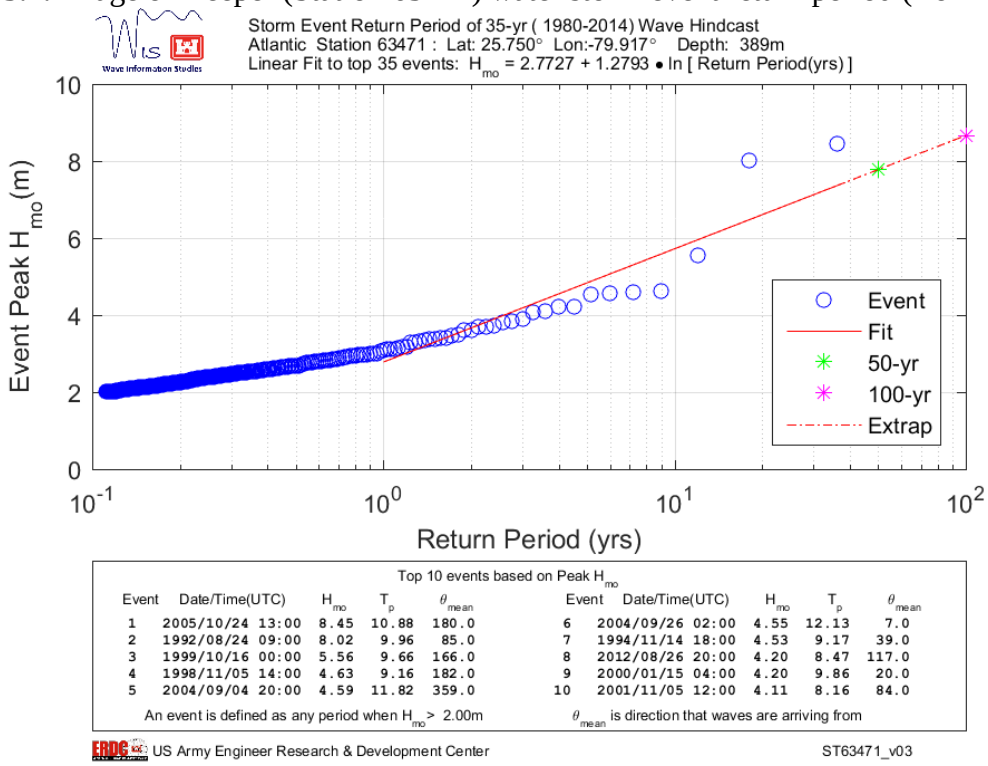


Figure 3.5. Image of Deep (Station 63471) water storm event return period (from WIS data).

Figure 3.6 and **Figure 3.7**show plots of winds roses off the coast of the City of Miami for deep water and shallow water conditions respectively.

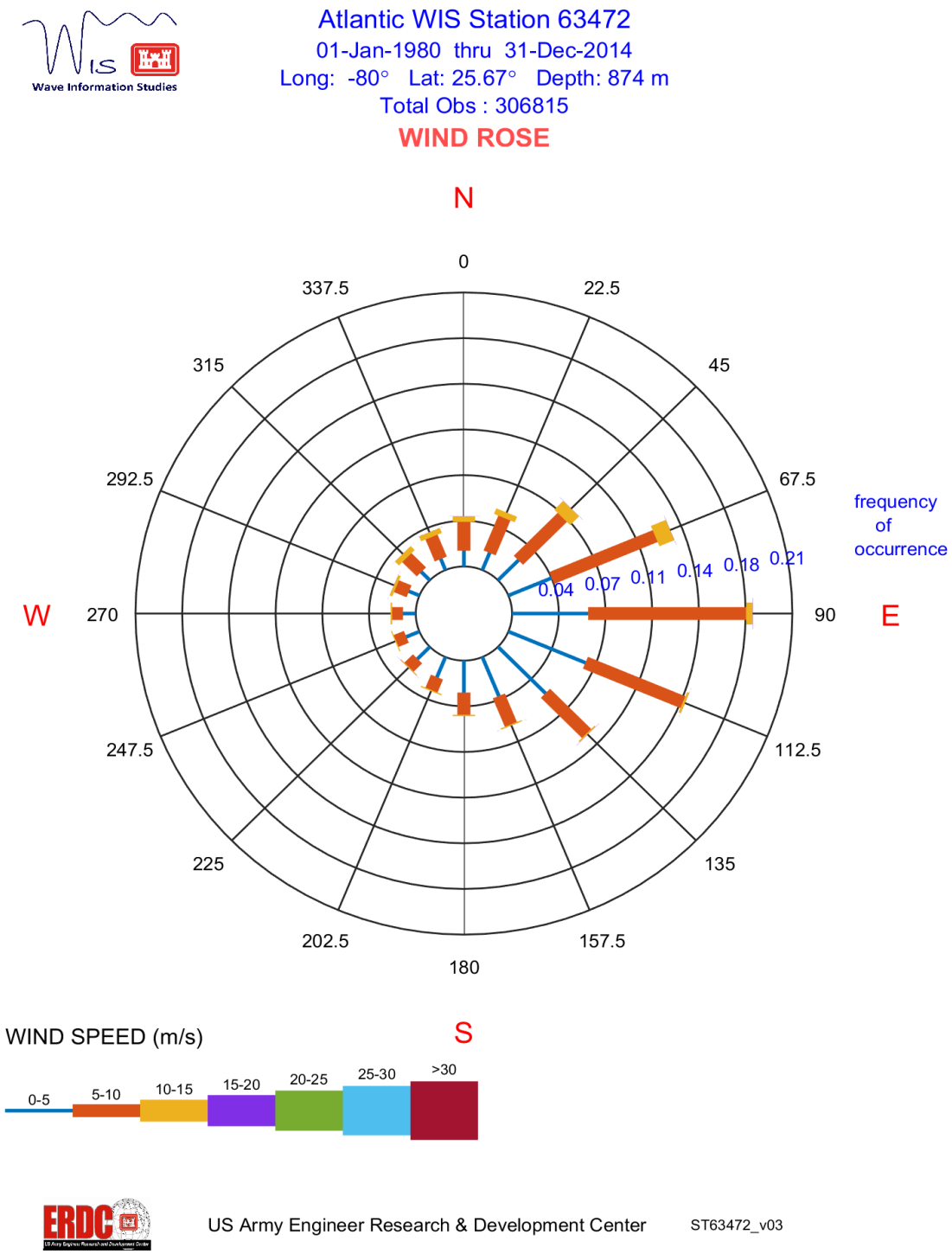


Figure 3.1. Image of Deeper Water Wind Rose (from WIS Data).

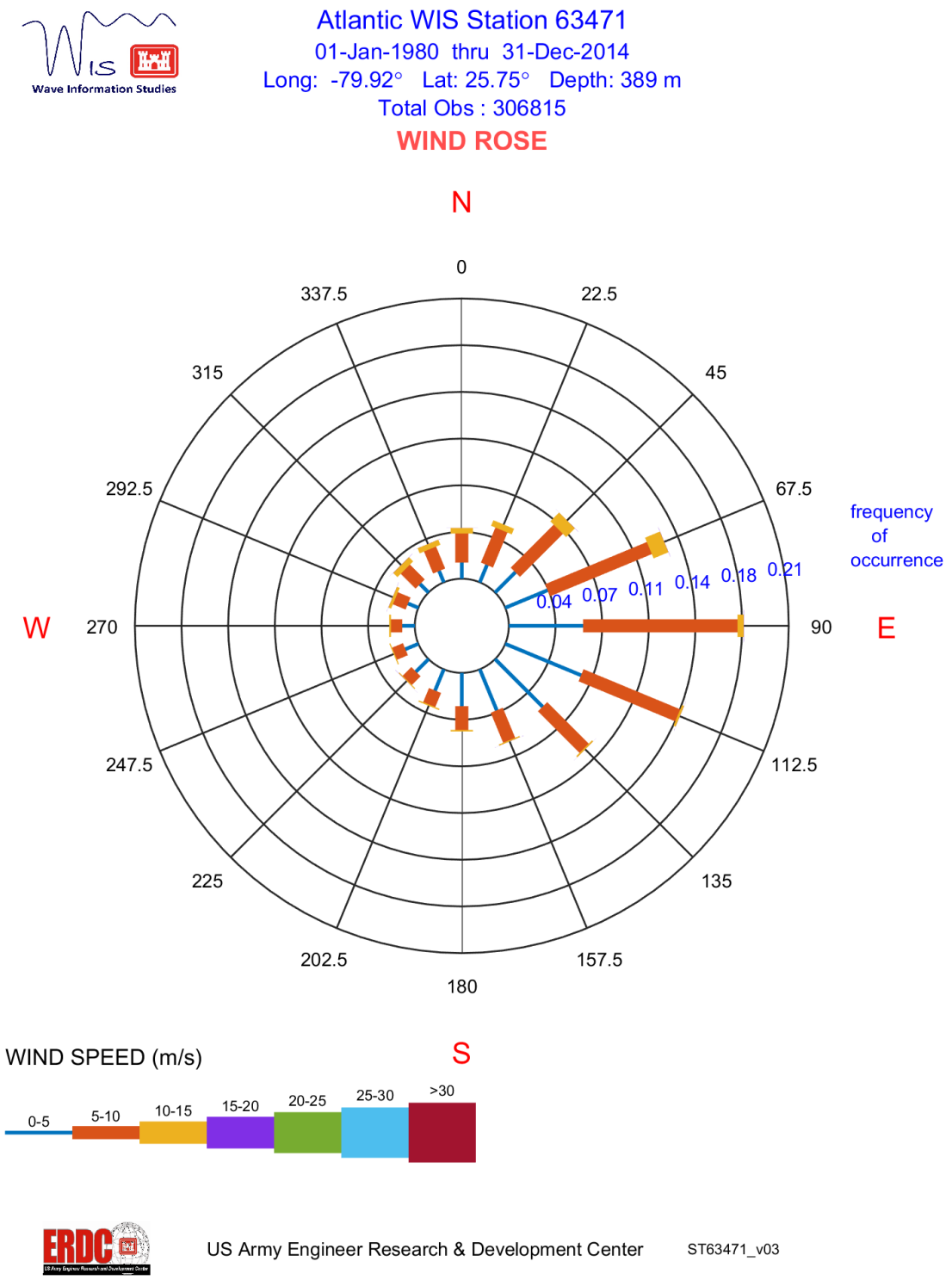


Figure 3.2. Image of Deep Water Wind Rose (from WIS Data).

3.4. ASTRONOMICAL TIDES & WATER LEVELS

3.4.1. ASTRONOMICAL TIDES

The most continuous data record (since 1970) available for sea level rise in the Miami-Dade County is the National Oceanographic and Atmospheric Administration (NOAA) Vaca Key, Florida Bay, FL tide gauge. A tide gauge is a device for measuring the changes in sea level relative to a vertical datum. The Vaca Key, Florida Bay, FL gauge (Station ID: 8723970) is located south of the Miami-Dade County in the Florida Keys near Marathon, FL. Another tide gauge available for tide water level information in Miami-Dade County is the National Oceanographic and Atmospheric Administration (NOAA) Virginia Key, Biscayne Bay, FL tide gauge (since 1994). The Virginia Key, Biscayne Bay, FL Gauge (Station ID: 8723214) is located east of Miami on the south end of Virginia Key on the Biscayne Bay side. (**Figure 3.8, Figure 3.9 & Figure 3.10**).

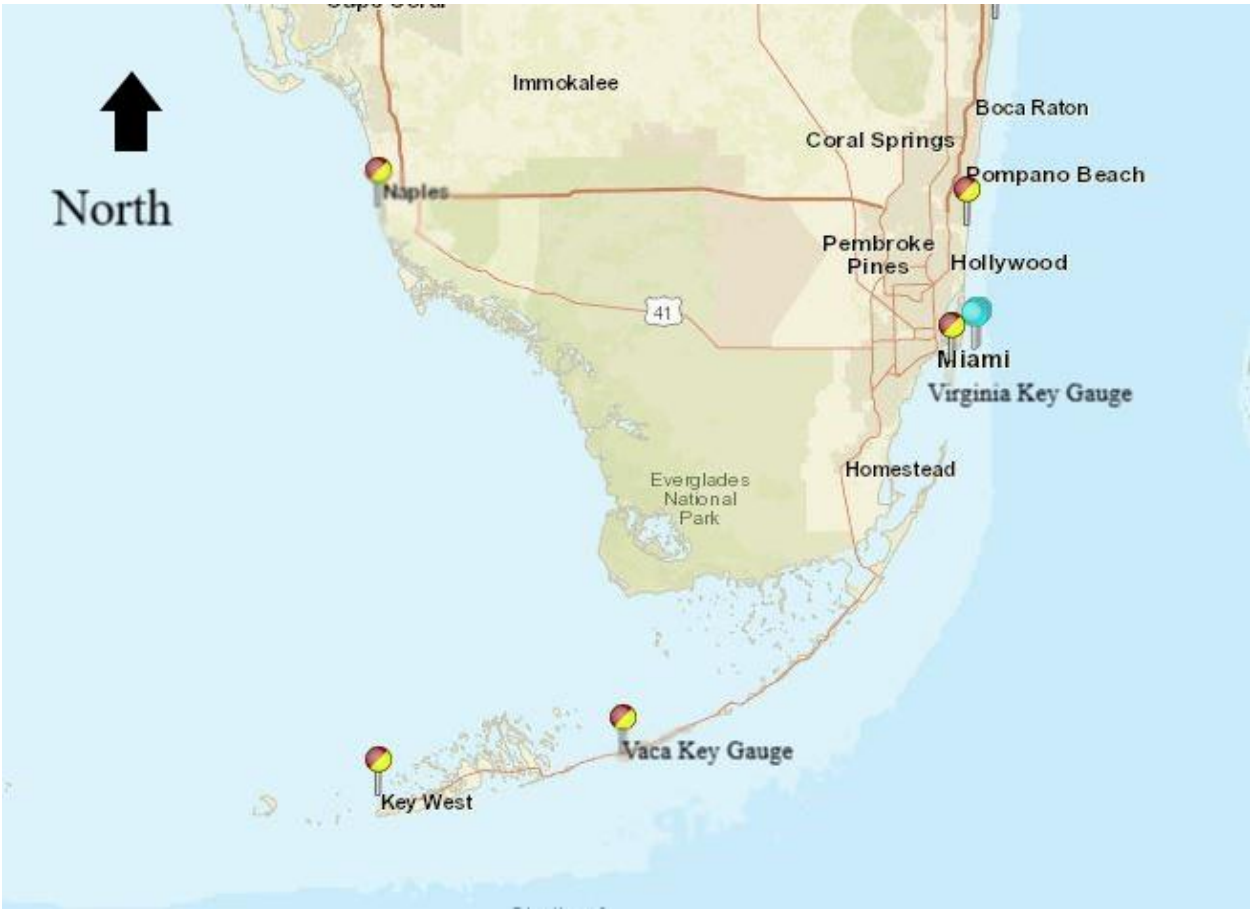


Figure 3.8. Map of NOAA Vaca Key Gauge and Virginia Key Gauge location in Florida.

Figure 3.9. Picture of NOAA Vaca Key Gauge.

Figure 3.10. Picture of NOAA Virginia Key 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 0.71 feet at the Vaca Key tide gauge (**Figure 3.11**) and 2.03 feet at the Virginia Key tide gauge (**Figure 3.12**).

The Virginia Key tide gauge is discussed because the location of this gauge is closer to the focus areas for the structural solutions proposed in this study. The Virginia Key gauge tide data will be referenced in the H&H Analysis of the Future With Project (FWP) and Future Without Project (FWOP) scenarios.

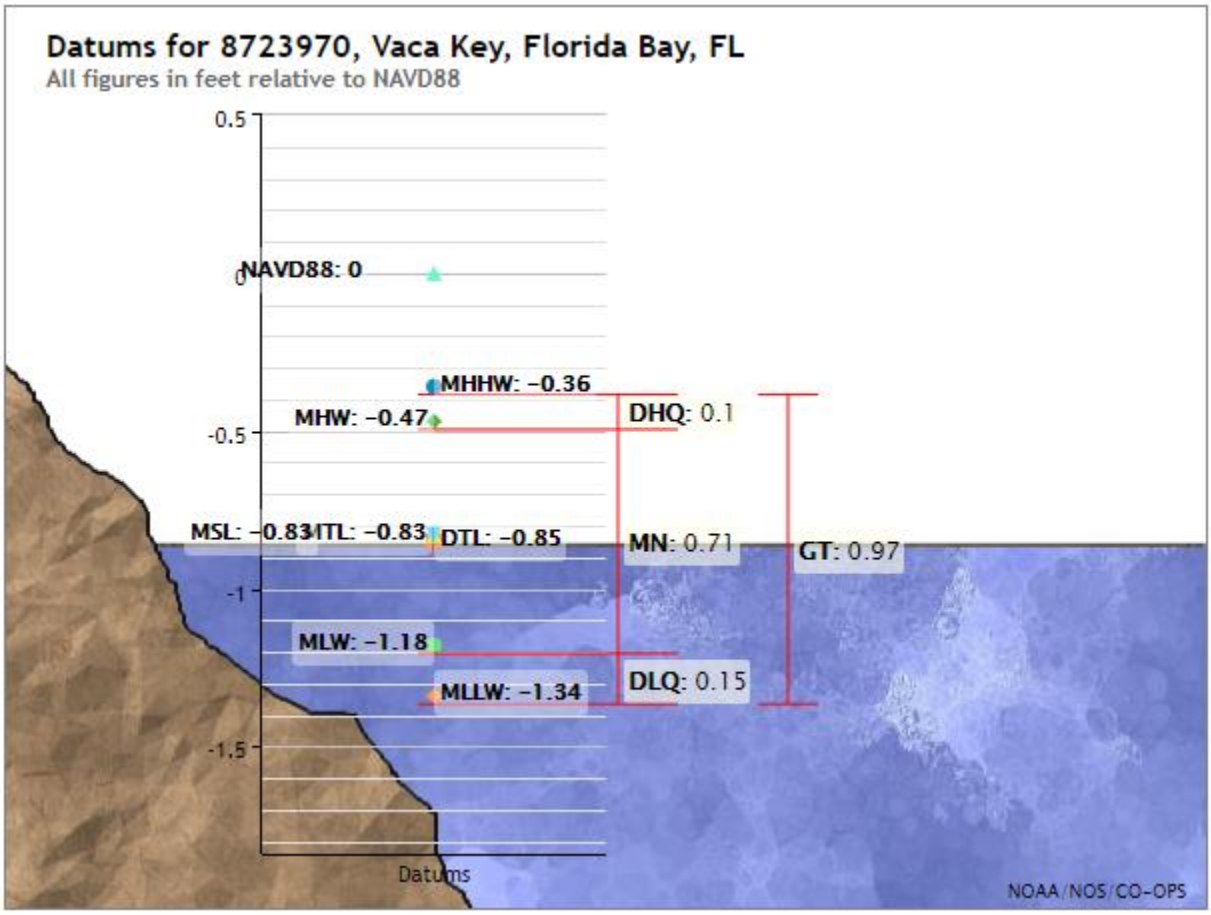


Figure 3.11. Vaca Key Gauge Image of the Tide Range.

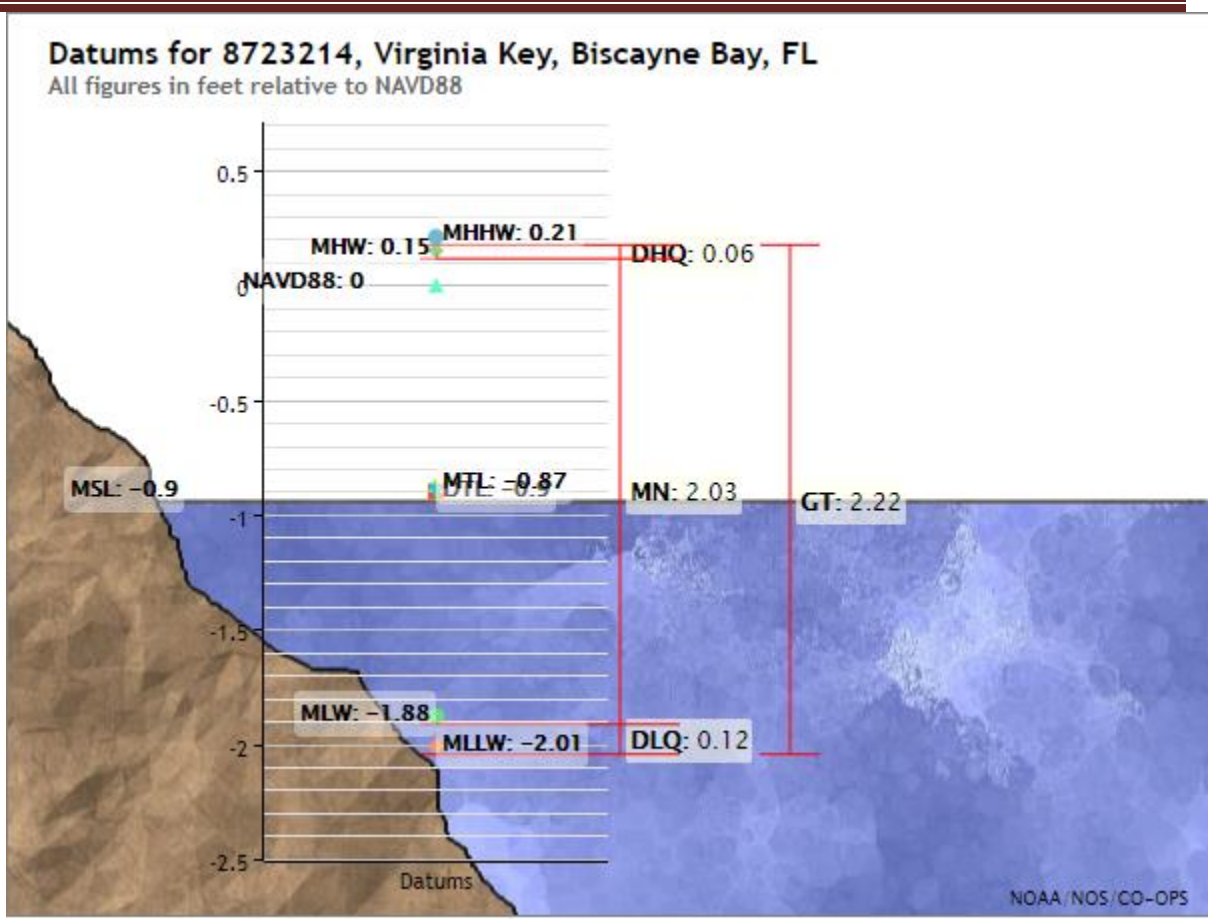
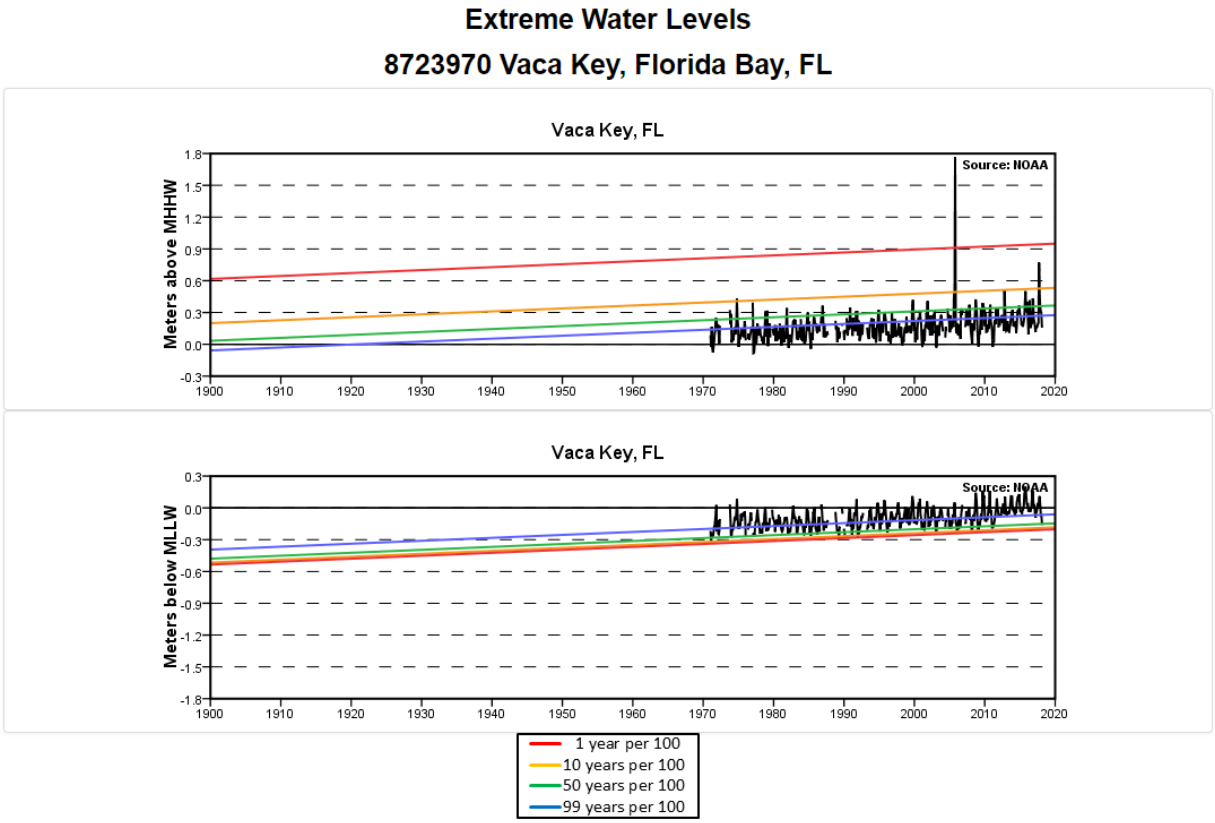


Figure 3.12. Virginia Key Gauge Image of the Tide Range.

3.4.2. WATER LEVELS

The tide gauge extreme water level records go back close to 50 years. **Figure 3.13** shows an image from the NOAA Tides and Currents website. The NOAA Vaca Key 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 monthly extreme water levels include a Mean Sea Level (MSL) trend of 2.78 millimeters/year with a 95% confidence interval of +/- 0.6 millimeters/year based on monthly MSL data from 1971 to 2006 which is equivalent to a change of 0.91 feet in 100 years.

The plots show the 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) datums established by CO-OPS (1 foot = 0.3 meters). On average, the 1% level (red) will be exceeded in only 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, although it could be exceeded more than once in other years.

Figure 3.13. Extreme Water Levels measured in Vaca Key Gage.

The Extreme Water Levels were not available for the Virginia Key gauge because the gauge does not have a 30 year period of record, which NOAA requires to perform a statistical analysis.

3.5. STORMS

Two general types of major storms affect the Miami-Dade County in the form of tropical storms and hurricanes.

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 of Gulf of Mexico and with high sustained 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 and sustained high winds (over 74 mph), heavy rainfall, large waves, tornadoes, 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 3.1**). This scale only relates to the hurricanes maximum sustained wind speed and does not reflect the high flooding impact potential associated with extreme rainfall produced by a hurricane, tropical storm or tropical depression, which has occurred in Miami-Dade County"

Table 3.1. 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

3.5.3. HISTORICAL STORMS

Miami-Dade County has experienced flooding from storms (tropical storms and hurricanes). The storms that impact the County are occurring more frequently and are more intense based on the historical record data. When describing a storm as being "more intense" one is referring to the characteristics of future storm events, such as the higher waves, larger storm surges and water surface elevations, greater winds, and potential heavier rainfall. Just like any previous storm events you could have one or more of the storm characteristics occurring during a storm event, which can and have previously reinforced each other leading to higher total water levels and flood-related damages.

Table 3.2 displays the date of historical storm events, the type of storm, and the peak water surface elevations if available. The peak water surface elevations shown were measured for storms occurring since 1970 from the NOAA Vaca Key tide gauge and since 1994 from the Virginia Key tide gauge. Older storm peak water surface elevations were gathered from previous reports and studies if available. All elevations are referenced to NAVD88. The list of historical storms, from gathering through research, has many other storms than what is shown in **Table 3.2**, but rainfall and tide data were difficult to locate. **Table 3.2** lists storms rating as being significant hurricanes, recorded rainfall over 3 inches (unless the tide or storm ranked above), and higher tides that were considered in the top ten for that tide gauge. The Great Miami Hurricane in 1926, the storm in **Table 3.2** with one of the highest water levels, ranks as one of the great storms for Miami-Dade County. Information about the 1926 hurricane was located in the National Weather Service archives. The Great Miami Hurricane storm track is shown in **Figure 3.16**. Hurricane Andrew's (1992) track is shown in **Figure 3.15**.

Hurricane Andrew (1992) illustrates the potential to experience high storm tides from powerful Hurricanes in the Southern part of Miami-Dade County. According to a National Hurricane Center report on Hurricane Andrew in reference to the storm tides, the following excerpt is from the report; "During the morning hour of 24 August, Andrew generated storm surge along shorelines of southern Florida (**Figure 3.14**). On the southeast Florida coast, peak storm surge arrived near the time of high astronomical tide. The height of the storm tide (the sum of the storm surge and astronomical tide, referenced to mean sea level) ranged from 4 to 6 feet in northern Biscayne bay increasing to a maximum value of 16.9 feet at the Burger King International Headquarters, located on the western shoreline in the center of the bay, and decreasing to 4 to 5 feet in southern Biscayne Bay. The observed storm tide values on the Florida southwest coast ranged from 4 to 5 feet near Flamingo to 6 to 7 feet near Goodland." In **Figure 3.14**, the value of 2.0 meters is shown at the mouth of the Miami River. That would be 6.56 feet Mean Sea Level (based on conversions at the Virginia Key gauge approximately 5.02 feet NAVD88).

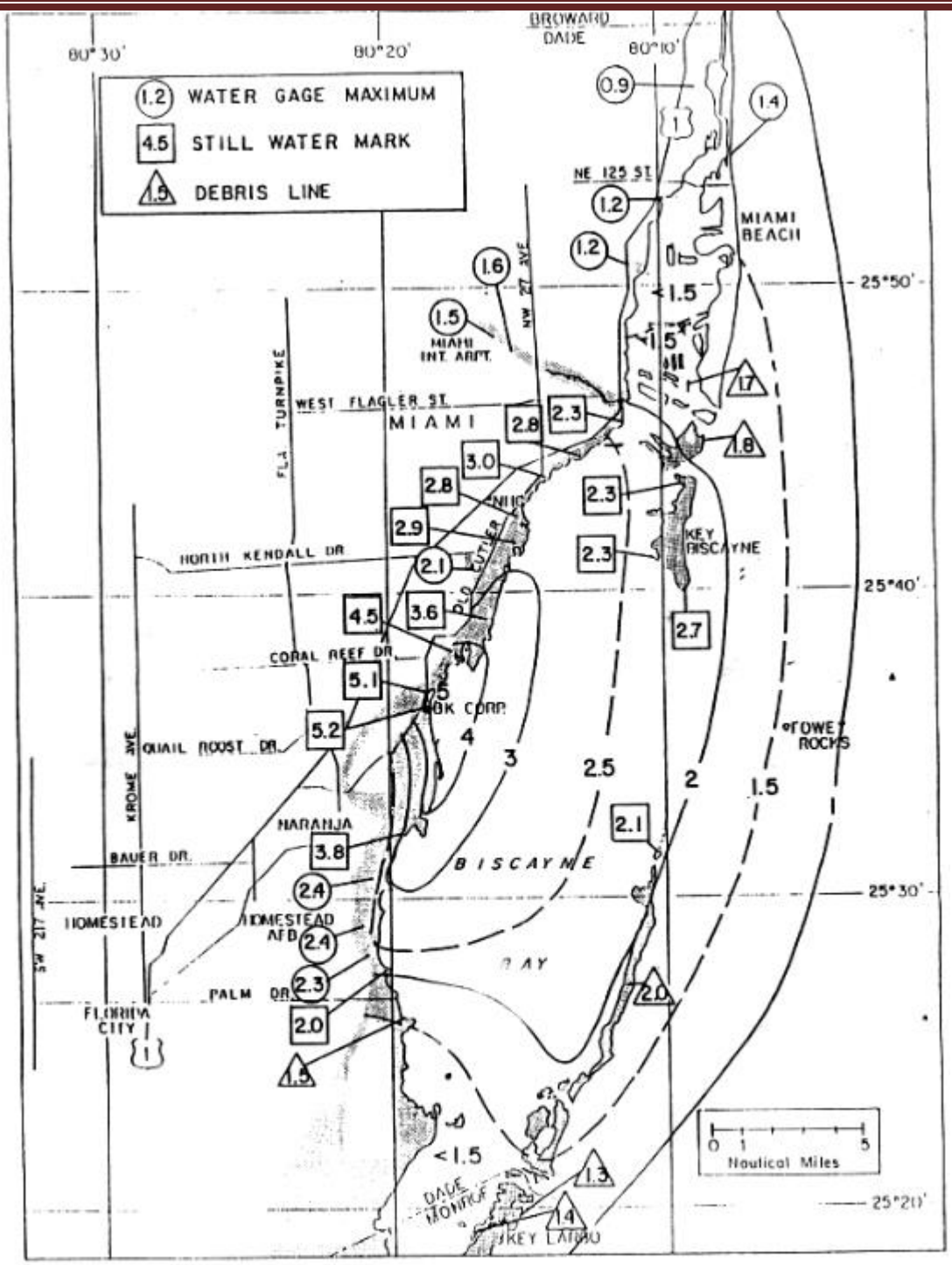


Fig. 7. Analysis and representative observations of the maximum storm tide during Hurricane Andrew's landfall in Florida, labelled in meters above 1929 mean sea level (1 meter = 3.28 ft). Shading indicates inland inundation.

Figure 3.14. Peak Tide Levels during Hurricane Andrew in 1992.

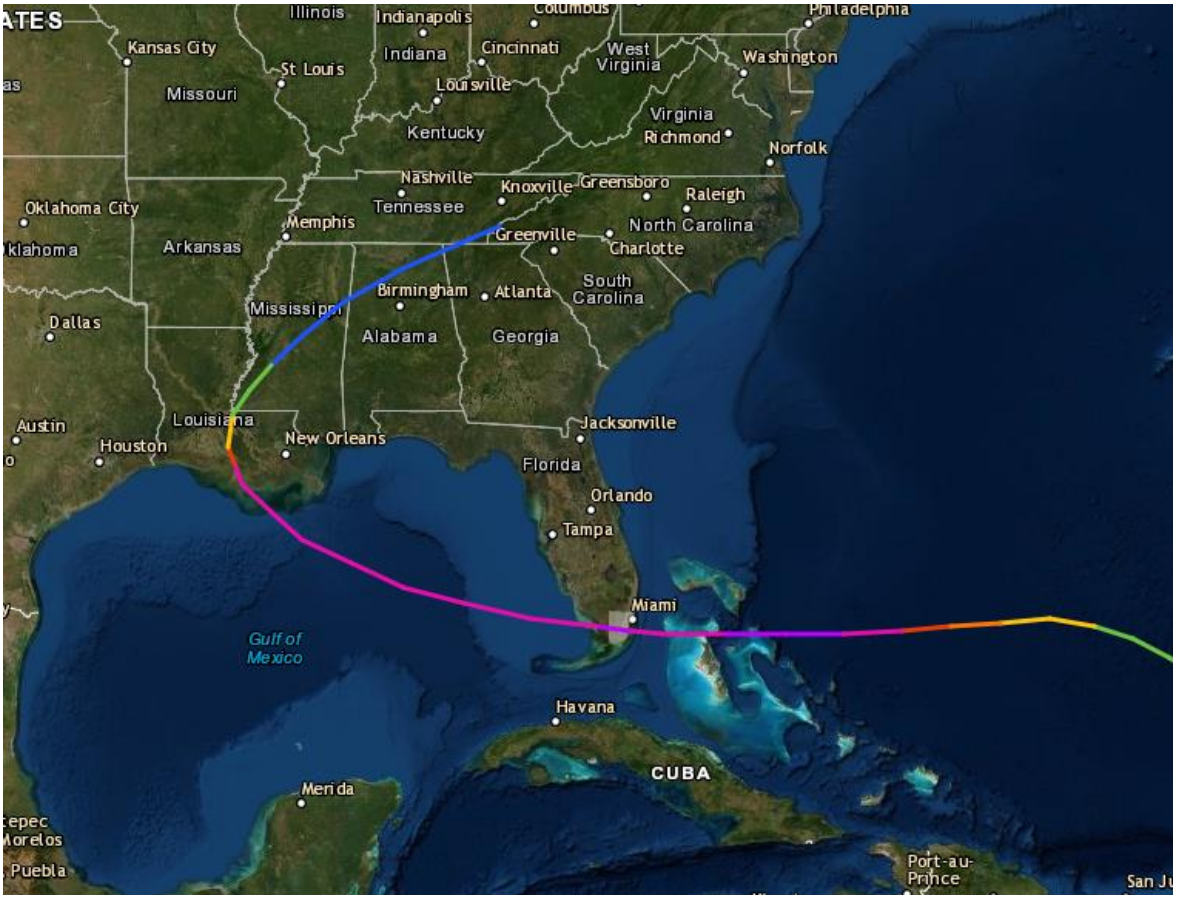


Figure 3.15. Hurricane Andrew Track in 1992.

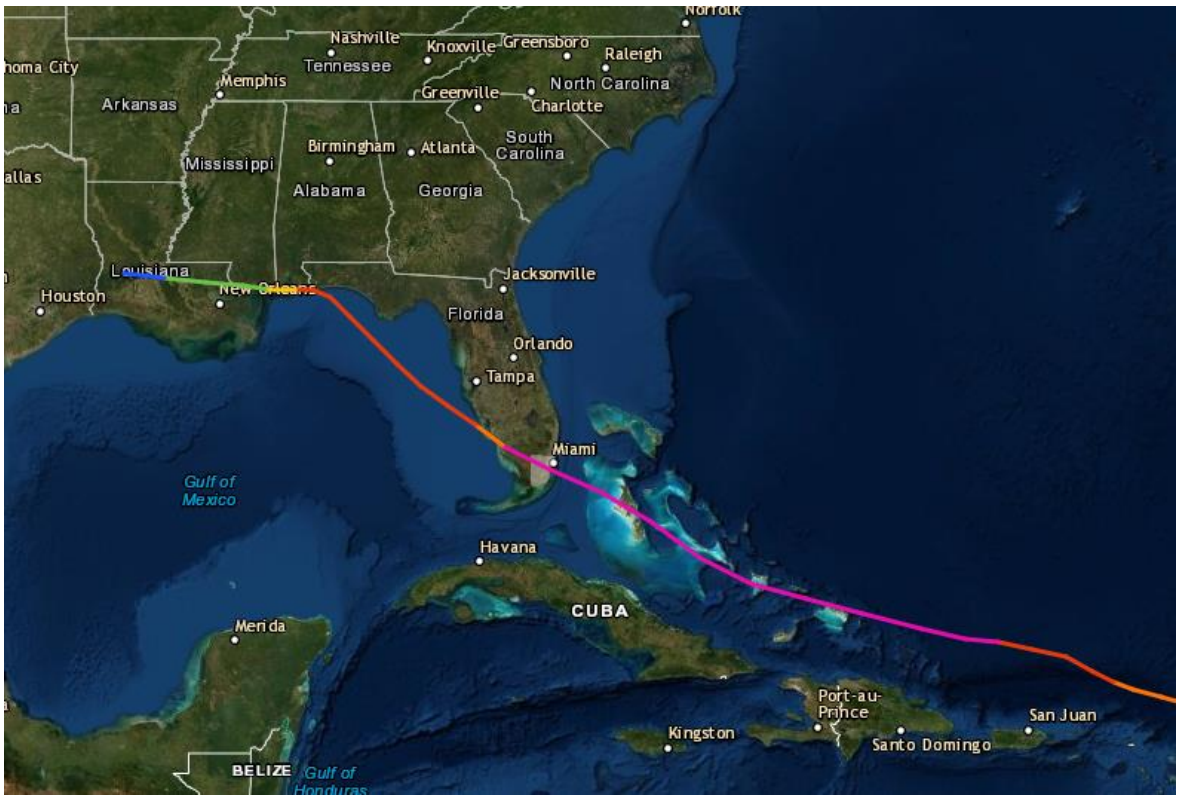


Figure 3.16. Great Miami Hurricane Track in 1926.

Table 3.2. Historical Storm Events

	Storm Event – Date & Name	Type of Storm	Peak Water Surface Elevations* > 0.0 feet NAVD 88 (in feet)	Rainfall (in inches) measured from Miami International Airport
1	August 16, 1888 (No Name)	Hurricane Cat 3	N/A	N/A
2	August 24, 1891 (No Name)	Hurricane Cat 1	N/A	N/A
3	October 17&20, 1904 (No Name)	Hurricane Cat 1	N/A	N/A
4	June 17, 1906 (No Name)	Hurricane Cat 1	N/A	4.75
5	October 18, 1906 (No Name)	Hurricane Cat 3	N/A	N/A
6	August 25, 1916 (No Name)	Tropical Storm	N/A	6.11
7	September 17-18, 1926 (Great Miami)	Hurricane Cat 4	7.1***	8.00
8	August 30, 1932 (No Name)	Tropical Storm	N/A	7.26
9	November 4, 1935 (No Name)	Hurricane Cat 2	N/A	N/A
10	September 15, 1945 (No Name)	Hurricane Cat 4	N/A	3.81
11	October 12, 1947 (No Name)	Hurricane Cat 1	N/A	N/A
12	October 6, 1948 (No Name)	Hurricane Cat 2	N/A	9.95
13	October 18, 1950 (King)	Hurricane Cat 4	N/A	1.97
14	August 24, 1992 (Andrew)	Hurricane Cat 5	5.66**	2.04
15	October 15, 1999 (Irene)	Hurricane Cat	1.02 (2.09)	6.81
16	October 4, 2000 (No Name)	Subtropical Depression	N/A	15.3
17	September 21, 2004 (Ivan)	Extratopical	0.52 (0.96)	0.01
18	August 25-26, 2005 (Katrina)	Hurricane Cat 1	1.19 (1.14)	5.10
19	September 20, 2005 (Rita)	Hurricane Cat 1	0.66 (2.15)	0.81
20	October 24, 2005 (Wilma)	Hurricane Cat 2	5.44 (2.77)	0.76
21	July 23, 2010 (Bonnie)	Tropical Storm	0.12 (0.38)	1.5
22	September 10, 2017 (Irma)	Hurricane Cat 3	2.19 (3.84)	5.18
23	August 24, 2019 (Erin)	Tropical Depression	0.57 (0.44)	0.07

* Vaca Key gauge (Virginia Key gauge) ** Data from a National Hurricane Center Report

*** Data from the National Weather Service Report

Figures 3.17 thru **3.20** are plots of the water surface elevation (the predicted versus the verified water levels) including the peak elevations measured at the NOAA – Vaca Key and Virginia Key gauges, during some of the storm events shown in **Table 3.2**.

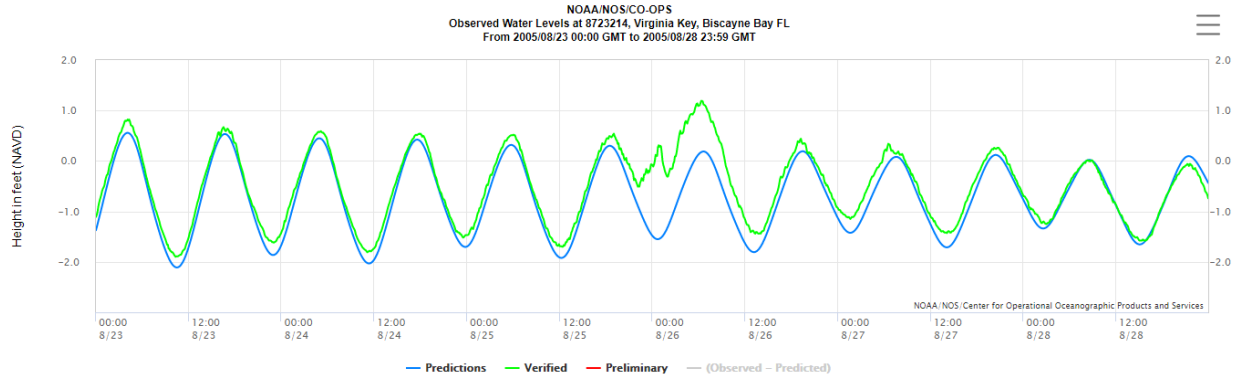


Figure 3.17. Predicted vs. Verified water levels during Hurricane Katrina in 2005 at Virginia Key gauge.

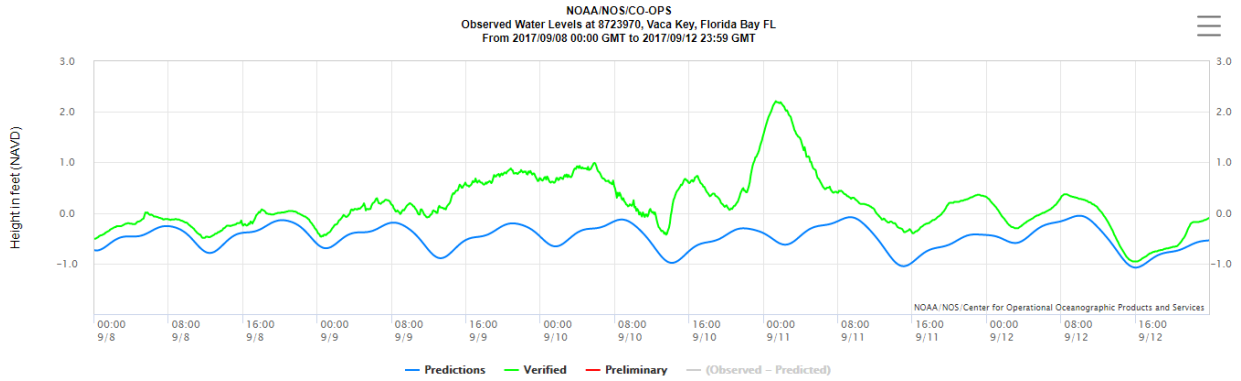


Figure 3.18. Predicted vs. Verified water levels during Hurricane Irma in 2017 at Vaca Key gauge.

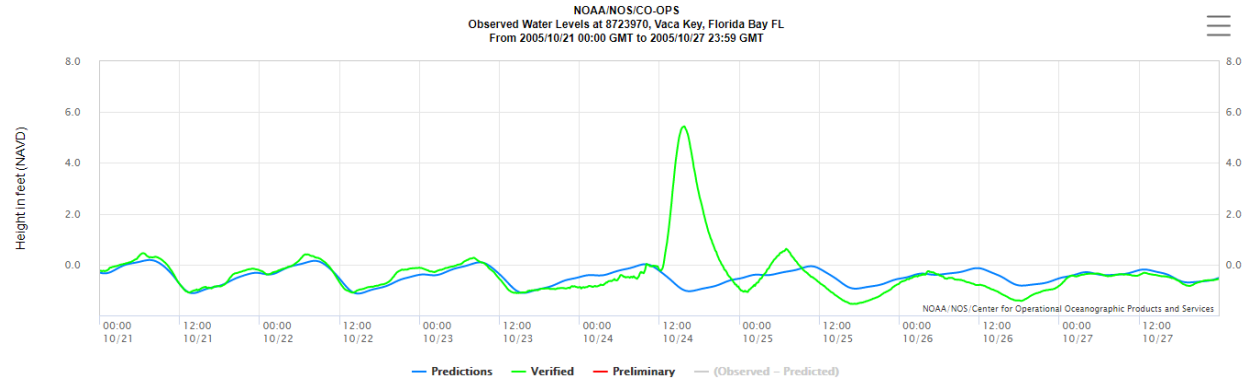


Figure 3.19. Predicted vs. Verified water levels during Hurricane Wilma in 2005 at Vaca Key gauge.

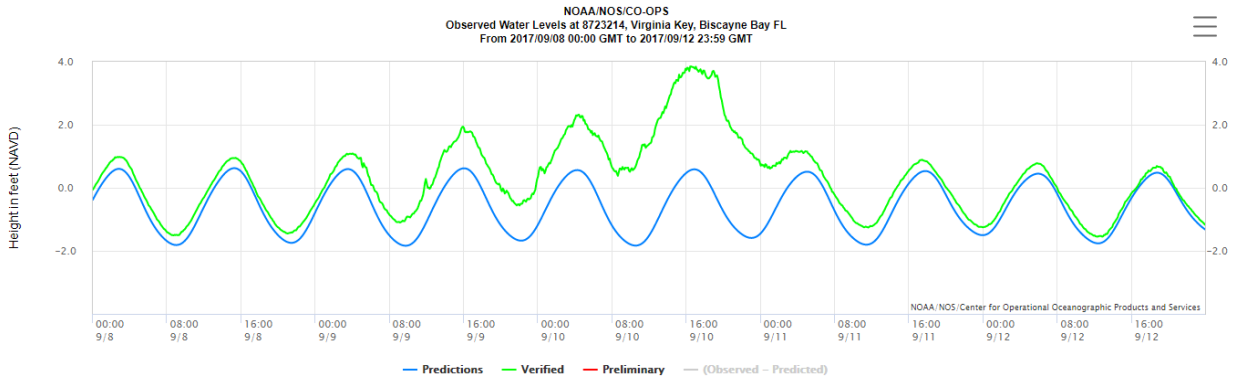


Figure 3.20. Predicted vs. Verified water levels during Hurricane Irma in 2017 at Virginia Key gauge.

3.6. SEA LEVEL RISE

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 Miami-Dade County averages 0.012 feet (3.6 mm) per year for the Vaca Key tide gauge **(Figure 3.21**). **Figure 3.21** shows a plot of the monthly mean sea level, at the NOAA – Vaca Key gauge from 1970 – 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.

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 2018 thru 2130 (adaptability year for 100 years after construction), for Miami-Dade County at the NOAA's Vaca Key Gauge, to be 0.84 feet NAVD88 for the low approach, 2.53 feet NAVD88 for the intermediate approach and 7.90 feet NAVD88 for the high scenario. See **Figure 3.22**.

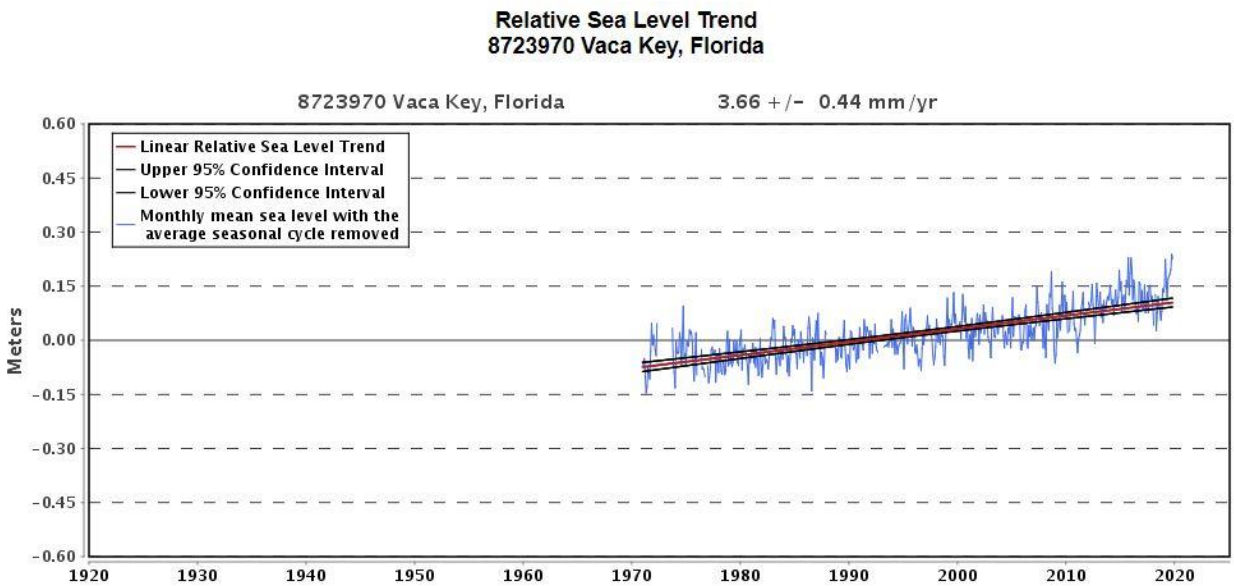


Figure 3.21. NOAA Sea Level Trend at Vaca Key

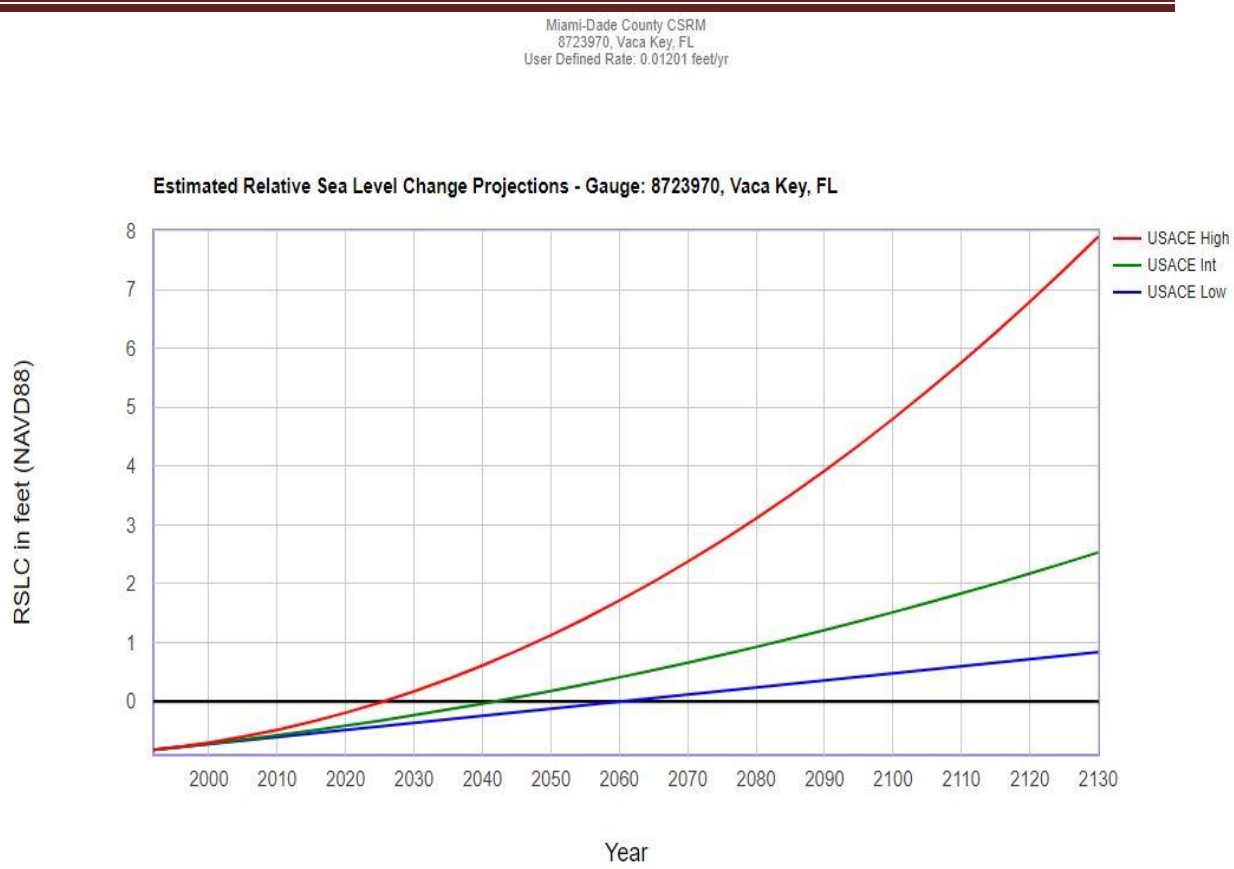


Figure 3.22. Modified NRC curves for predicting future rates of RSLR.

More detail on the tides and SLR and their influence on water levels in Miami-Dade County will be discussed in the HH&C Sub Appendix.

3.6.2. LAND SUBSIDENCE

Miami-Dade County does not generally experience land subsidence.

CHAPTER 4 FEMA SOUTH FLORIDA STORM SURGE STUDY (SFLSSS)

This chapter briefly discusses the FEMA Region IV South Florida Storm Surge Study (SFLSSS) and how the study results were used for the Miami-Dade County Back Bay CSRM Study. The SFLSSS is discussed in more detail in the HH&C Sub Appendix at the end of this Engineering Appendix.

4.1. PURPOSE OF SFLSSS STUDY

The FEMA SFLSSS includes the coastal counties of Miami-Dade, Broward, Palm Beach, and Monroe (See **Figure 4.1**). The purpose of the study is to determine the flood risk from 50% (2 year), 20% (5 year), 10% (10 year), 4% (25 year), 2% (50 year), 1% (100 year), and 0.2% (500 year) annual exceedance probability (AEP) floods for these coastal areas for production of revised Flood Insurance Rate Maps (FIRMs). AEP 1% (or other %) is further defined as an annual chance or 1% (or other %) chance to occur in any one year.

4.2. STORMS AND WATER

To develop the South Florida (SFL) 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 SFL coast. Extra-Tropical storms were not included in the FEMA SFLSSS because previous FEMA studies in Florida provided evidence that they do not contribute to AEPs10% (10 year), 2% (50 year), 1% (100 year), 0.5% (200 year), 0.2% (500 year), and 0.1% (500 year) storm surge heights. As a result from modeling computed for this study, storm surge, wave height, wave period, and water levels were produced for frequencies at save points located throughout Miami-Dade County.

It should be noted that the native datum of the FEMA SFLSSS 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 SFLSSS water levels were referenced to the year 1992, the midpoint of the

current National Tidal Datum Epoch (NTDE) (1983-2001). The values were also converted from MSL to NAVD88.

For the H&H analysis the following discussion of the adjustments to the 1992 SFLSSS water levels are listed below:

The difference or increase in the relative local level rise (based on the USACE low scenario) from 1992 to 2018 was added to bring the water levels to present date (start of this study). The difference or increase in the relative local level rise (based on the USACE high scenario) from 2018 to 2030 was added to bring the water levels to 2030 (end of construction and start of the economic period of analysis). The difference or increase in the relative local level rise (based on the USACE high scenario) from 2030 to 2079 was added to bring the water levels to 2079 (end of the economic period of analysis for a 50 year project life). The total waters levels (including tides, storm surge, and SLR) at year 2079 were used for the structural solution analysis.

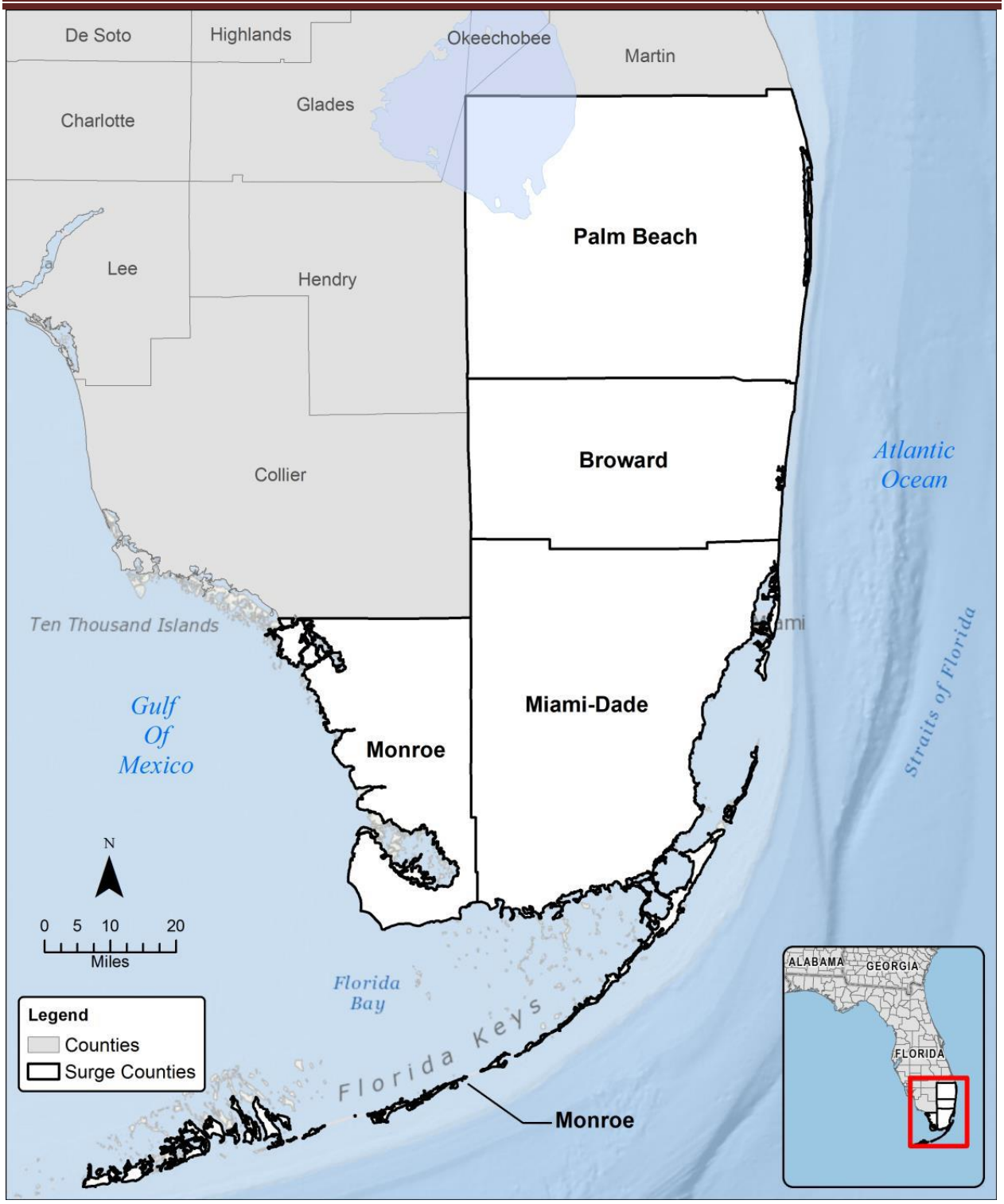


Figure 4.1. Map of Counties included in SFLSSS area (image courtesy of BakerAECOM report).

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 Miami-Dade County Back Bay CSRM. More detail can be read in the Engineering Sub Appendices.

5.2. WATER LEVELS

Water levels were evaluated at several save points (locations within the model output that correspond to an approximate area for the proposed structural measures) for structural solutions from the SFLSSS modeling output located in Biscayne Bay in Miami-Dade County (Figure 5.1). Table 5.1 shows the FEMA SFLSSS Total Stillwater level (SWL) elevations for year 2079 for different save points. The table shows four AEPs (5% (20 year), 2% (50 year), 1% (100 year), and 0.5% (200 year)) which were chosen to be analyzed. This assumes relative sea level rise to year 2079 based on the USACE high curve. The SFLSSS 1% AEP (100 year) SWL elevation was the water level chosen to design the project alignment for this study. The project delivery team (PDT) decided that the 1% AEP SWL elevation should be the value used in the range of elevations for formulating separable measures throughout Miami-Dade County. Water Levels and the projected rates of Sea Level Rise are discussed in more detail in the HH&C Sub Appendix at the end of this Engineering Appendix.

Table 5.1 shows the 2079 (end of the economic period of analysis) SWL elevations from the 90% confidence limit including SLR using the USACE high curve. The tentatively selected plan (TSP) and the project alignment (the Edgewater portion is now deleted) is based off of the 1% Peak SWL elevations from the 90% Confidence Limit (CL) with RLSR in the year 2079 in **Table 5.1**.

Table 5.1. SFLSSS Peak SWL elev. (90 CL) (in feet, NAVD88) w/ RLSLR at 2079 (High SLR Curve).

Annual Recurrence Interval (years)	Annual Exceedance Probability (AEP) in %	**Miami River (Save Point 443)**	**Edgewater (Save Point 572)**	**Little River (Save Point 614)**	**Biscayne Canal (Save Point 621)**
20	5	7.66	7.15	7.09	7.18
50	2	9.38	8.43	8.33	8.53
100	1	10.6	9.44	9.24	9.48
200	0.5	11.95	10.73	10.17	10.41

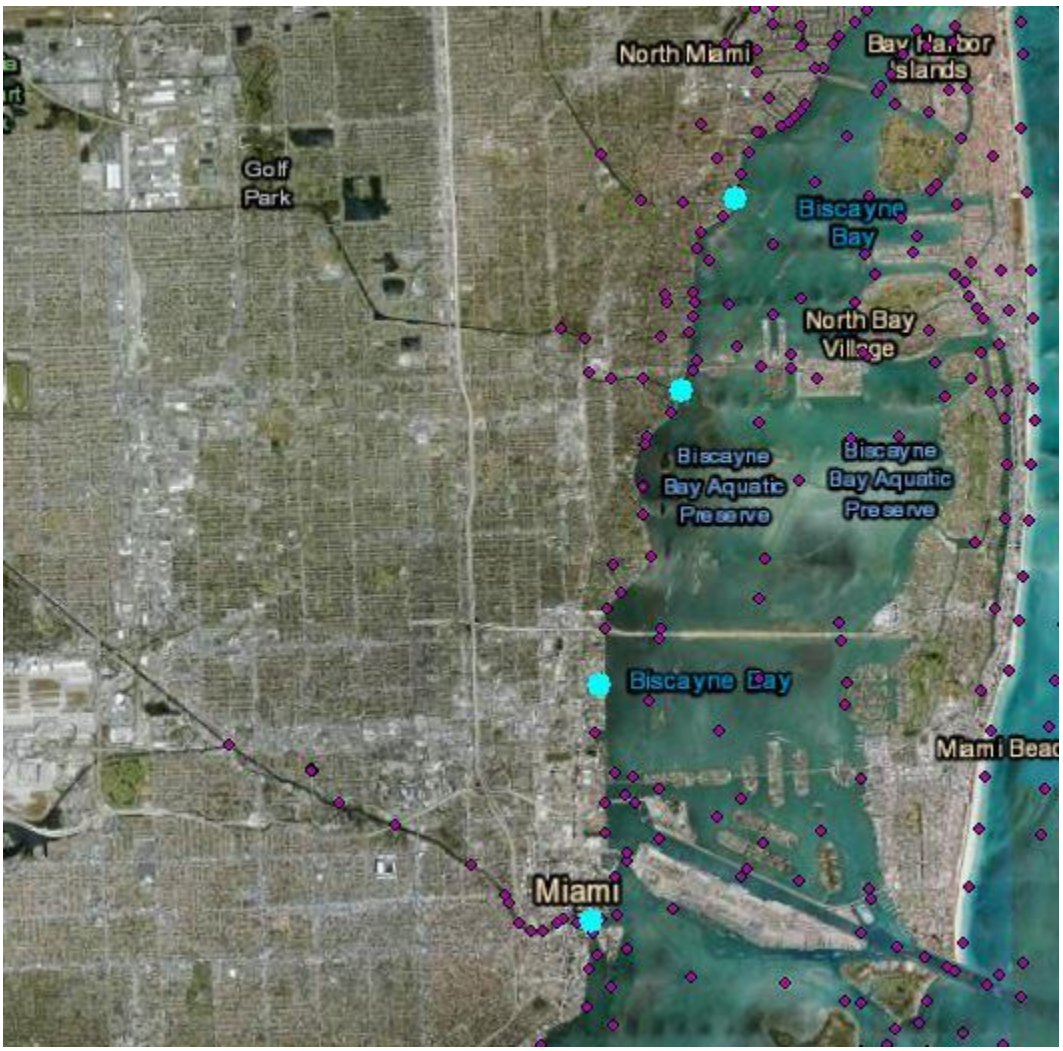


Figure 5.1. SFLSSS model save points for the four watershed areas in Miami-Dade County (the purple circles in the image – the blue nodes are the saved points used for the four focus areas)

5.3. PROJECT ALIGNMENT

The term "Project Alignment" refers to structural methods (floodwall, surge barrier gates, etc.) that serve as a barrier to water flow. The project alignment may vary per water level frequency. Estimates of locations and footprints of the structural measures have been initially determined based on the USACE derived 2079 1% AEP SWL elevation from the FEMA South Florida Storm Surge Study (includes tides, storm surge, and USACE high curve sea level rise) and will be finalized during the Preconstruction, Engineering, and Design Phase of the project when more detailed surveys and data are available.

Figure 5.2 is an image of the Future With Project conditions from the 1% AEP SWL elevations. **Figure 5.3** is an image of the Future Without Project conditions from the 1% AEP SWL elevations. The HH&C Sub Appendix includes more discussion and images of the different modeled scenarios.

Update: The paragraphs below discuss Alternative 7 mentioned in the main report. Alternative 7 is not the TSP. Alternative 8 is the current TSP and does not include the Edgewater floodwall alignment. Any discussions about the Edgewater alignment are to be ignored. A revision of the Engineering Main Appendix and the Sub Appendices will be provided before the Final Report.

The Main Report mentions Alternative 7 and it includes the proposed structural measures that will be discussed below. The proposed alignment has structural solutions including, footprints of proposed pump stations, floodwalls, and storm surge barriers. The methodology in determining this alignment was based on the 1% AEP 2079 SWL elevation, derived from the SFLSSS data discussed in the HH&C Sub Appendix, at first look for the TSP. The recommended plan will involve the economics team optimizing the benefits for other AEPs (5%, 2%, and 0.5%) and determining the greatest net benefits for the National Economic Development (NED) plan.

The derived 1% AEP SWL elevation was mapped using the HEC-RAS software to assess the extents of the 1% AEP SWL elevations. The extent of the 1% SWL elevation influenced the length of the floodwall. If the AEP is more than 1% there is a chance the length of the alignment

will be shortened and lowered. The top of floodwall elevation varies and is shown in **Tables 5.4 to 5.7**.

The intent of the Miami River alignment is to provide the maximum protection, which is primarily located as close to the existing shoreline as possible, to the watersheds. Portions of the Floodwall are shown in the water a distance from the current bulkhead. Structural engineers suggest the new floodwall not replace the current bulkhead due to unknown construction of the existing bulkhead. Major complications from removing the existing bulkhead could cause instabilities in the current properties near the existing bulkhead. Recommended practice is to encapsulate the existing bulkhead and build as close as possible to the existing bulkhead without disturbing the foundation. A floodwall would be impermeable and include a cutoff wall that would extend on the flood side below the base of the floodwall. Bulkheads are not as impermeable as a concrete floodwall. The current estimation of the cutoff wall length would need to be assessed in the PED phase. According to the Geotechnical Sub Appendix, some groundwater flow may seep under the cutoff wall and the proposed design addresses that limitation.

The floodwall base requires a large footprint below the Biscayne Bay bottom to provide structural support to the stem. The survey of the existing bulkhead will determine the support extents of the existing sheet pile and/or concrete pilings. **Figure 5.1a** is a cross section of the proposed design of the floodwall in the water. In the proposed design, a distance from the existing bulkhead to the new floodwall base and/or stem is dependent upon the surveyed condition of the bulkhead. Coordination of the new floodwall support and the existing bulkhead support has to occur in order to be at a minimal distance without disturbance. Along the length of the floodwall it is estimated that the stem height of the wall (the portion visible) could minimally be 20 feet tall from the existing Biscayne Bay bottom. The space between the new floodwall and the existing bulkhead would be backfilled and capped with concrete creating a sidewalk that would be at the same elevation of the current bulkhead. On the flood side of the floodwall from the stem to approximately 5 feet beyond the foundation base and about 1 foot below the 4 foot thick base foundation will be rip rap to protect the floodwall from scour. The scour analysis will finalize the actual distance of the rip rap.

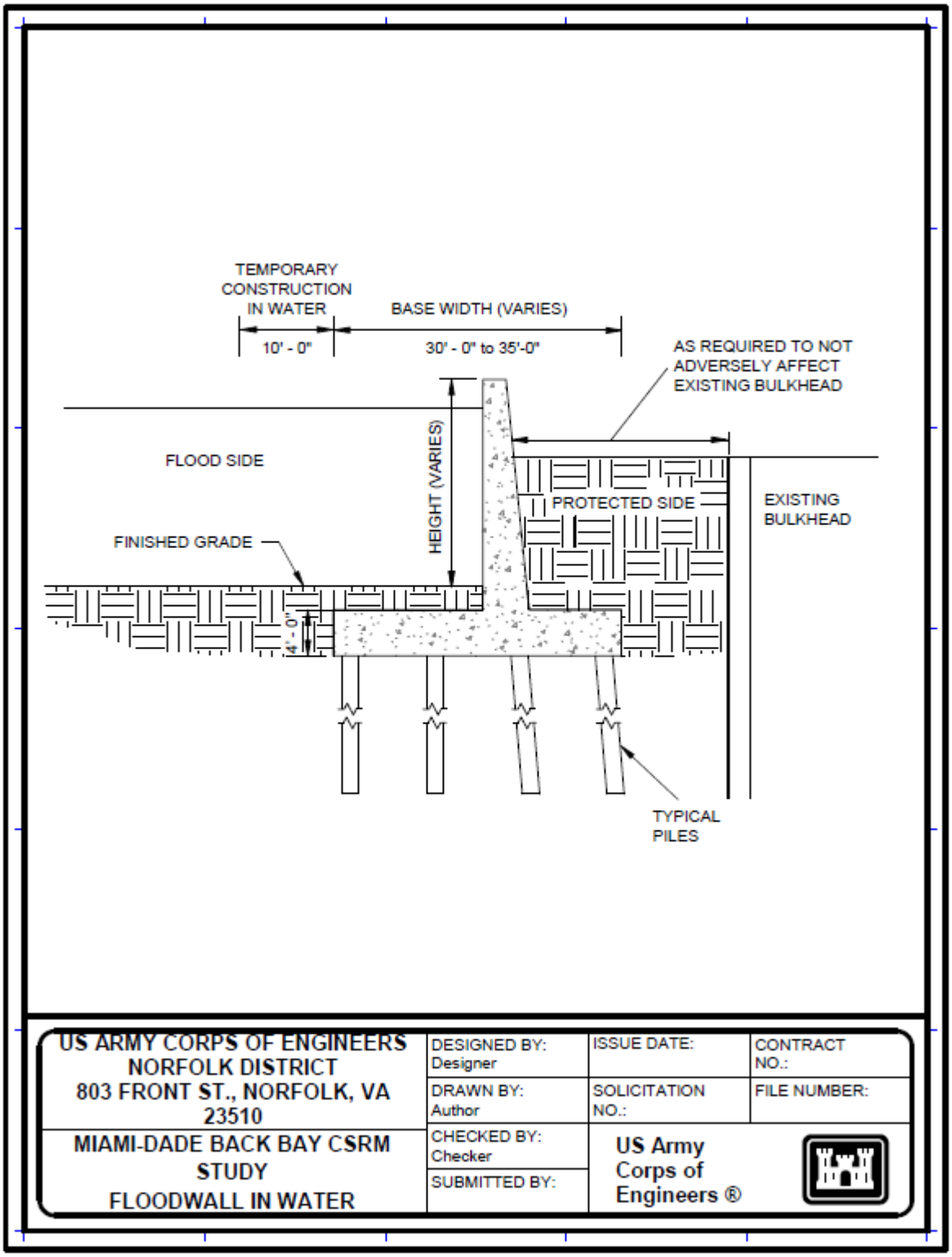


Figure 5.1a. Proposed cross section of floodwall in water.

The project alignments also include storm surge barriers. Chapter 5 Section 5.5 discusses the different types of closure barriers. For the Miami River alignment, a sector gate is currently

proposed at the mouth of the river. The sector gate orientation with a possible inclusion of forward pumps has challenges associated with the design. A proposed solution is under current development and will be presented in the Final Report. For the Little River and Biscayne watersheds miter gate closures are proposed. The Little River and Biscayne watersheds have challenges associated with locating the closure gates at the mouth of the canals due to multiple outlets of the canal at the mouth. Providing a closure at the mouth of the Little River and the Biscayne Canal would require a continuous line of protection and tie-in to higher ground. The location of both closures proposed at Biscayne Blvd. provides the higher ground tie in necessary for a continuous line of protection.

The proposed structural alignments may be discussed in various sections in the Engineering Appendices where appropriate.

The following project alignment descriptions are as follows:

- Focus area of Miami River watershed. The current alignment begins on SW 15th Rd near the intersection of Brickell Ave. and continues east towards Biscayne Bay. The transition to the Biscayne Bay occurs and turns north and continues along the shoreline to the mouth of the Miami River Crossing the Miami River is where a proposed Sector Gate and possibly a Miter Gate will be configured to allow boat traffic. The wall continues north on land entering on Biscayne Blvd. The floodwall will follow Biscayne Blvd primarily on the east side to 13th St. where the floodwall will turn left and end at NE 2nd Ave. For the TSP, a pump station location either integrated with the Sector Gate or located off Brickell Ave. behind the First Presbyterian Church in the parking lot.

- Focus area of Edgewater watershed. (now deleted) The floodwall starts on 13th at the intersection with Biscayne Blvd. and turns east onto N. Bayshore Dr. until it exits into Biscayne Bay near NE 20th Terrace. The floodwall continues north along the existing shoreline until north of Martell Park and then turns west and ends west of NE 5th Ave. The intent of this alignment is to provide maximum protection to the Edgewater watershed. Two proposed pump stations with locations are NE 22nd St and N Bayshore Dr. and at the end of NE 34th St. This watershed is not part of the TSP.

- Focus area of Little River watershed. The floodwall starts on Biscayne Blvd at NE 61st St. and heads north on Biscayne Blvd. and continues to the canal bridge where there will be a storm surge barrier gate. On the north side of the canal the floodwall continues north on Biscayne Blvd. for a short distance and then turns east on NE 78th St. and continues north on NE 7th Ave. until E. Dixie Hwy. The floodwall continues on E. Dixie Hwy north to NE 10th Ave and continues north and turns west on NE 91st Terrace. The floodwall continues west on NE 91st Terrace and turns north on NE 9th Ave to NE 93rd St and turns west and ends at NE 9th Ave. The proposed pump station location is on the west side of Biscayne Blvd next to the canal.

- Focus area of Biscayne Canal watershed. The floodwall starts where the Little River Floodwall ends on NE 93rd St. and NE 9th Ave. The floodwall continues north on NE 9th Ave to NE 95th St. The floodwall continues west on NE 95th St. to Biscayne Blvd. and continues on Biscayne Blvd north to Biscayne Canal where there will be a storm surge miter gate across the canal. The floodwall continues north on Biscayne Blvd to west on Chase Bank for a short distance before heading north on NE 13th Ave. The floodwall continues north on NE 13th Ave towards NE 112th St and continues west and ends at NE 12th Ave. The proposed pump station location is on the west side of Biscayne Blvd next to the canal.

- Arch Creek Watershed. Floodwalls may be proposed, but at this time no modeling analysis has been done to suggest one. Suggest larger pump stations be constructed to relieve the existing low lying areas as a result of the 1% AEP SWL elevation. The proposed pump station location would be either to upgrade the existing pump stations, AC1 and AC2, or upgrade the G58 water control structure.

The sizing of the pump capacities and pump stations at this time has yet to be determined. The sizing will most likely be based off of the existing assessment reports on the C4 and C7 watersheds. More discussion on the HH&C analysis is in the HH&C Sub Appendix.

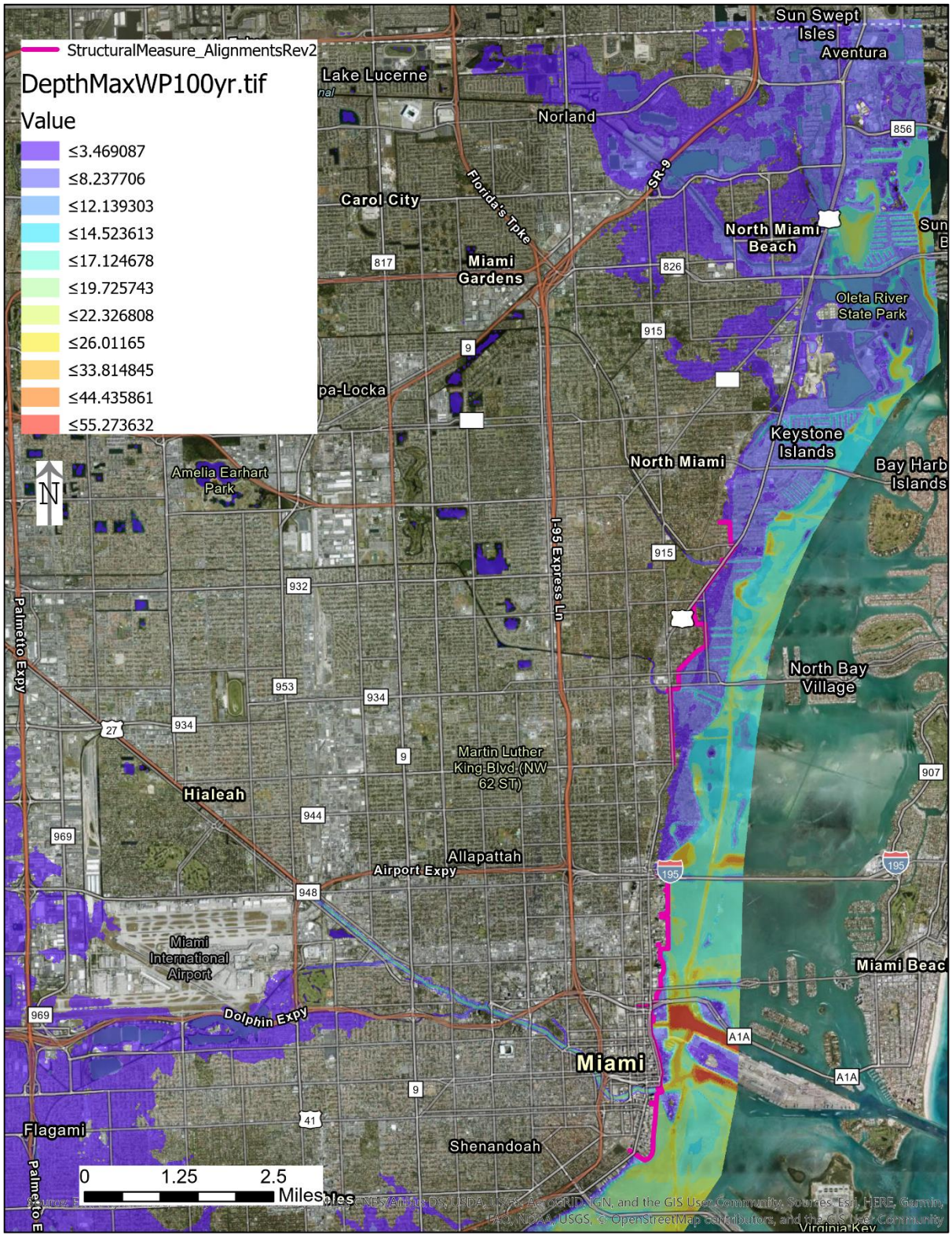


Figure 5.2. 1% AEP 2079 Future With Project Alignment.

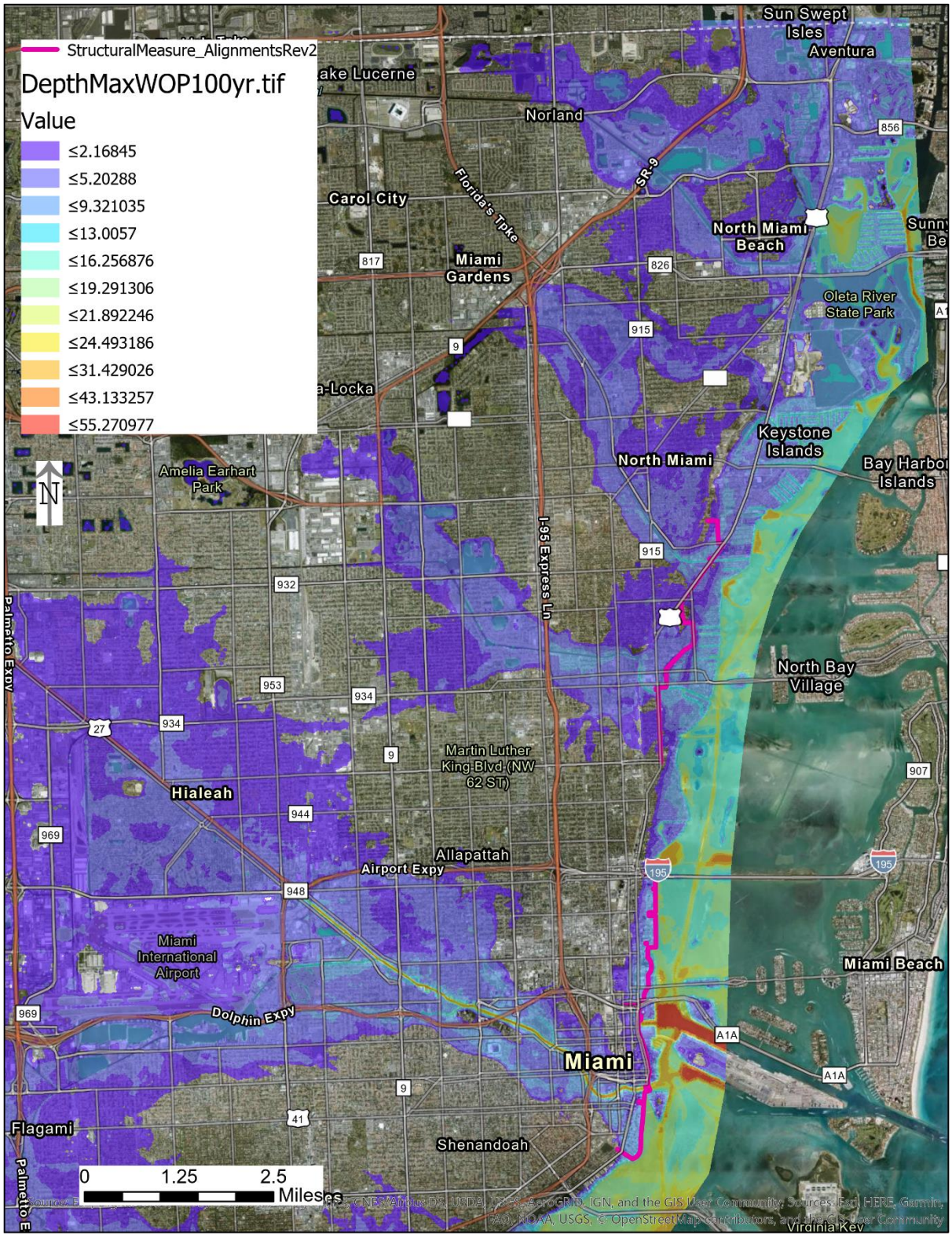


Figure 5.3. 1% AEP 2079 Future Without Project Alignment (alignment shown for reference).

It should be noted that there are environmental concerns of the project alignment in the water in the Biscayne Bay. Reference the Main Report and Appendix D for the details about the environmental concerns. Note that non-structural measures are also being reviewed for this study and are not reflected in this Engineering Appendix.

5.4. GENERATION II COASTAL RISK MODEL (G2CRM)

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 surge 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.

5.4.1. METEROLOGICAL DRIVING FORCES

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 Miami-Dade County, all 390 storms from the FEMA SFLSSS data were included in the 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. 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 the U.S. Army Corps of Engineers Engineering Research and Development Center (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 reaches to refine and further develop the model.

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. For Miami-Dade County, the Norfolk District coordinated with the Jacksonville District team who used Ocean Weather, Inc. (OWI) hindcast storm data to develop their Tropical and Extra- Tropical storm seasons. The Extra-Tropical seasonal storm probabilities are calculated from the historical storms derived from the OWI hindcast storm data covering 1973 to 2016, while the tropical storm seasons were calculated from OWI hindcast storm data covering 1925 to 2017. **Table 5.2** displays the storm seasons for Miami-Dade County. Though the seasons were analyzed, it was determined that extra-tropical storms (for storm surge events) do not contribute to damage producing events and therefore were not included in the G2CRM analysis.

Table 5.2. Miami-Dade County Storm Season

Storm Season	**Start Date**	**End Date**	**Extra-Tropical**		**Tropical**	
			Number	Probability	Number	Probability
Extratropical Winter/Spring	Dec 1	Apr 30	16	0.36	0	0.00
Tropical Early Summer	May 1	Jul 31	0	0.00	1	0.01
Tropical Peak	Aug 1	Sep 30	0	0.00	17	0.18
Extratropical/Tropical	Oct 1	Nov 30	2	0.05	11	0.12

The combination of the plausible storm dataset and the specified storm season allows the G2CRM 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 and was specified as 15 days. Based on this interval the model attempts to place subsequent storm events outside of a 15 day window surrounding the date of the previous storm (i.e. a minimum of 7.5 days prior to the storm event and a minimum of 7.5 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.

5.4.2. 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 10 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. FLOODWALL

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 for both land based and submerged floodwalls varied from 2 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 5.6** shows a graphical image of a typical I-wall with sheet pile base. **Figure 5.7** 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 a maximum of 20 feet in length. While the floodwall protects against storm surge, the porous geology provides a path for floodwater to travel underneath the wall. To assist in combatting this issue, a seepage barrier has been incorporated in conjunction with the floodwall. 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 5.8** shows a graphical image of a typical T-wall above ground and foundation below ground. **Figure 5.9** 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.

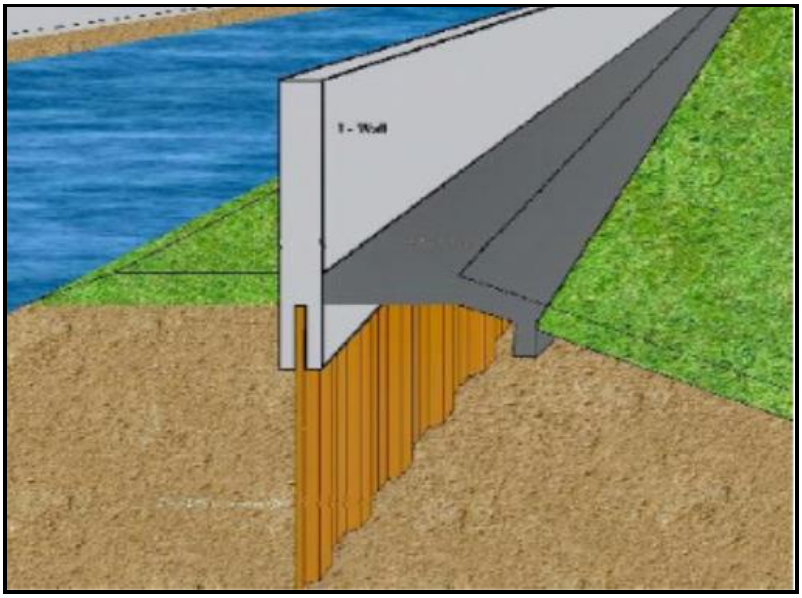


Figure 5.6. Graphical drawing of a typical sheet pile-based I-wall cross section (Image courtesy of Tulane University)

Figure 5.7. Existing I-wall located in the City of Norfolk.

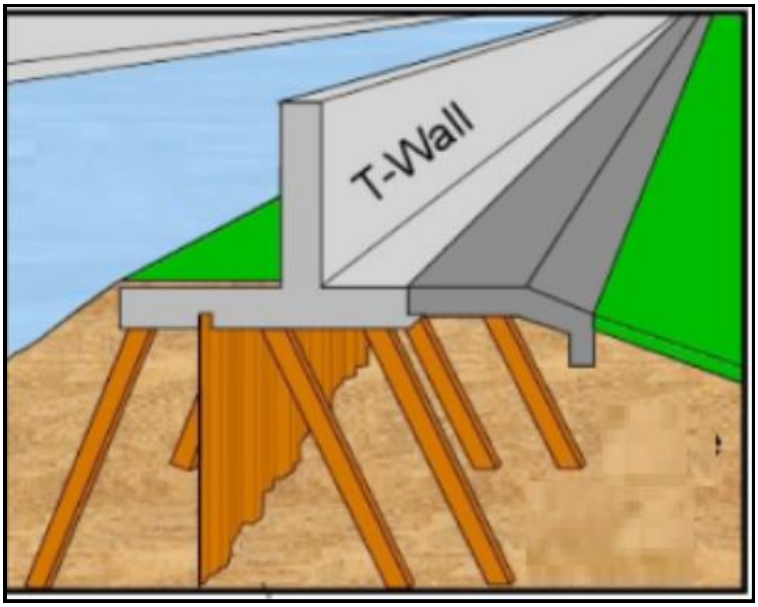


Figure 5.8. Graphical drawing of a Typical T-wall cross-section (Image courtesy of Tulane University).

Figure 5.9. Existing T-wall located at the Richmond Flood Control Project

5.6. 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 108 locations identified for potential road gate closures for the inland alignments (Error! Not a valid bookmark self-reference.**,** Error! Reference source not found.**,** Error! Reference source not found., and **Table 5.6**). 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 heights of the closures were all approximately 3 feet to 17 feet above grade. We extrapolated estimates of construction from the as-built drawings of the existing flood gates used in downtown Norfolk. Error! Reference source not found. and Error! Reference source not found. are images of a gate closure that is a part of the Norfolk Floodwall.

Table 5.3. Miami River Alignment Proposed Gate Closures.

		PRELIMINARY STREET CLOSURES MIAMI-DADE COUNTY BACK BAY CSRM 10% DESIGN – MIAMI RIVER ALIGNMENT				
No.	**Location**	**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
1	SE 15th Rd. Median	0+75	6	13.4	7.4	55
2	SE 15th Rd. & Brickell Ave Intersection	3+35	-0.5	13.4	13.9	90
3	Brickell Bay Dr.	11+60	-2.5	13.4	15.9	60
4	Brickell Key Dr.	48+00	-4.0	13.4	17.4	65
5	Brickell Avenue	10+30	-6.0	13.4	19.4	90
6	(2x) Circular Entrance	69+25	-1.0	15.5	16.5	25
7	Private Entrance	70+20	-2.0	15.5	17.5	40
8	(3x) Private Entrance	71+40	-1.0	15.5	16.5	50
9	Chopin Plaza	75+50	-0.5	15.5	16.0	70
10	Park Entrance 1	81+35	1.0	15.5	14.5	65
11	Park Entrance 2	84+95	3.0	15.5	12.5	20
12	Park Entrance 3	88+40	2.5	15.5	13.0	30
13	Park Drive Thru 1	91+95	3.5	15.5	12.0	40
14	Park Drive Thru 2	95+55	3.0	15.5	12.5	50
15	Miamarina Pkwy Dr. 1	100+95	2.0	15.5	13.5	50
16	Miamarina Pkwy Dr. 2	102+95	0.5	15.5	15.0	60
17	Railroad Crossing	104+55	2.0	15.5	13.5	20
18	NE 8th Street	110+15	1.0	15.5	14.5	45
19	WWTP Entrance 1	114+80	0	15.5	15.5	35
20	WWTP Entrance 2	121+40	1.0	15.5	14.5	25
21	Park Entrance 4	123+60	-1.0	15.5	16.5	25
22	Private Entrance	125+40	-2.0	15.5	17.5	55
23	Ramp to Bridge	128+25	-3.0	15.5	18.5	50
24	Biscayne Blvd.	130+60	-3.0	15.5	18.5	90
25	NE 2nd Ave.	136+00	2.0	15.5	13.5	50

Table 5.4. Edgewater Alignment Proposed Gate Closures.

		PRELIMINARY STREET CLOSURES MIAMI-DADE COUNTY BACK BAY CSRM 10% DESIGN – EDGEWATER ALIGNMENT				
No.	**Location**	**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
	NE 2nd Ave.	5+00	2.0	13.6	11.6	50
	Biscayne Blvd.	10+40	-2.5	13.6	16.1	90
	N Bayshore Dr. Entrance Ramp	16+70	-2.5	13.6	16.1	40
1	NE 13th Street	17+75	-2.0	13.6	15.6	55
2	NE 14th Street	22+55	-1.0	13.6	14.6	50
3	Private Entrance	25+55	-1.0	13.6	14.6	35
4	NE 15th Street	28+55	-2.5	13.6	16.1	60
5	NE 16th Street	22+35	-1.5	13.6	15.1	30
6	Private Entrance	33+30	-1.0	13.6	14.1	35
7	Private Entrance	36+25	-2.0	13.6	15.6	25
8	Private Entrance	38+60	-2.0	13.6	15.6	20
9	Park Entrance	44+00	0	13.6	13.6	90
10	Park Entrance	47+15	0	13.6	13.6	30
11	Park Entrance	50+50	-1.5	13.6	15.1	50
12	NE 36th Street Bridge	121+15	-6.0	15.4	21.4	60

Table 5.5. Little River Alignment Proposed Gate Closures.

		PRELIMINARY STREET CLOSURES MIAMI-DADE COUNTY BACK BAY CSRM 10% DESIGN – LITTLE RIVER ALIGNMENT				
No.	**Location**	**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
1	NE 62nd Street	2+00	7.0	10.0	3.0	70
2	NE 63rd Street	4+60	7.0	10.0	3.0	55
3	NE 64th Street	8+20	6.0	10.0	4.0	70
4	NE 64th Terrace	10+30	5.0	10.0	5.0	45
5	NE 66th Street	13+85	5.0	10.0	5.0	80
6	NE 67th Street	16+60	5.5	10.0	4.5	50
7	NE 68th Street	18+95	6.0	10.0	4.0	40
8	Private Entrance	20+15	6.0	10.0	4.0	15
9	NE 69th Street	21+95	4.5	10.0	5.5	80
10	Private Entrance	23+90	4.0	10.0	6.0	25
11	NE 70th Street	24+85	3.5	10.0	6.5	40
12	Private Entrance	25+30	4.0	10.0	6.0	35
13	Private Entrance	26+10	3.0	10.0	7.0	30
14	NE 71st Street	27+50	4.0	10.0	6.0	75
15	NE 72nd Street	29+95	5.0	10.0	5.0	75
16	(2x) Private Entrance	30+75	5.0	10.0	5.0	25
17	NE 72nd Terrace	32+60	4.0	10.0	6.0	70
18	Private Entrance	33+80	4.0	10.0	6.0	30
19	NE 73rd Street	35+45	4.0	10.0	6.0	65
20	(2x) Private Entrance	36+85	5.0	10.0	5.0	30
21	NE 74th Street	38+30	5.0	10.0	5.0	60
22	NE 75th Street	41+20	6.0	10.0	4.0	60
23	Private Entrance	42+95	5.0	10.0	5.0	15
24	NE 76th Street	44+30	4.0	10.0	6.0	50
25	Private Entrance	45+20	3.5	10.0	6.5	30
26	NE 77th Street	47+45	2.0	10.0	8.0	55
27	(2x) Private Entrance	51+40	3.5	10.9	7.4	30
28	Private Entrance	52+85	0.5	10.9	10.4	30
29	Private Entrance	53+90	-1.0	10.9	11.9	25
30	Private Entrance	55+45	-2.0	10.9	12.9	30
31	Private Entrance	57+40	-2.0	10.9	12.9	30
32	Private Entrance	59+10	-2.0	10.9	12.9	20
33	NE 79th Street	60+75	-2.0	10.9	12.9	70
34	Private Entrance	61+60	-2.0	10.9	12.9	25
35	NE 80th Street	63+80	0	10.9	10.9	70
36	NE 81st Street	66+40	3.5	10.9	7.4	60
37	NE 82nd Street	69+10	5.0	10.9	5.9	50
38	(2x) NE 82nd Terrace	72+60	4.0	10.9	6.9	40
39	NE 83rd Street	75+65	6.5	10.9	4.4	45
40	NE 84th Street	82+90	5.0	10.9	5.9	45
41	Unnamed Street	85+30	4.0	10.9	6.9	40
42	NE 85th Street	87+85	2.0	10.9	8.9	75
43	NE 86th Street	91+30	0	10.9	10.9	60
44	E Dixie Hwy	96+80	3.0	10.9	7.9	25
45	NE 88th Street	99+55	1.5	10.9	9.4	70
46	NE 89th Street	104+00	2.0	10.9	8.9	70
47	NE 89th Terrace	106+05	0	10.9	10.9	55
48	NE 90th Street	109+15	0	10.9	10.9	90
49	NE 91st Terrace	119+10	7.0	10.9	3.9	30
50	NE 92nd Street	122+40	8.0	10.9	2.9	50

Table 5.6. Biscayne Canal Alignment Proposed Gate Closures.

		PRELIMINARY STREET CLOSURES MIAMI-DADE COUNTY BACK BAY CSRM 10% DESIGN – BISCAYNE CANAL ALIGNMENT				
No.	**Location**	**Station**	**Sill Elevation (ft.)**	**Elevation of Top of Gate (ft.)**	**Height (ft.)**	**Width (ft.)**
1	NE 96th Street	13+25	4.0	10.6	6.6	80
2	NE 96th St to Biscayne Blvd Exit	14+00	4.0	10.6	6.6	40
3	NE 10th Avenue	22+70	2.0	10.6	8.6	50
4	NE 105th Street	49+55	2.0	11.6	9.6	130
5	Towerside Terrace	51+80	1.0	11.6	10.6	45
6	NE 107th Street	57+20	1.0	11.6	10.6	85
7	NE 107th St to Biscayne Blvd Exit	58+10	1.0	11.6	10.6	40
8	Private Entrance	58+90	1.0	11.6	10.6	60
9	Private Entrance	59+60	0.5	11.6	11.1	70
10	Biscayne Blvd	61+00	1.0	11.6	10.6	70
11	Private Entrance	64+10	-1.0	11.6	12.6	60
12	NE 108th Street	64+70	-1.0	11.6	12.6	35
13	NE 109th Street	68+40	-2.5	11.6	14.1	80
14	Private Entrance	69+65	-3.0	11.6	14.6	25
15	NE 110th Street	71+40	-3.0	11.6	14.6	65
16	NE 110th Terrace	74+40	-2.0	11.6	13.6	70
17	NE 111th Street	77+40	-1.0	11.6	12.6	55
18	NE 112th Street	80+80	3.0	11.6	8.6	20
19	(2x) Private Entrance	84+30	3.0	11.6	8.6	25
20	Private Entrance	85+25	3.0	11.6	8.6	100
21	Private Entrance	86+30	4.0	11.6	7.6	25

Figure 5.10. Bird's eye view of a gate closure apart of the existing Norfolk Floodwall.

Figure 5.11. A gate closure apart of the existing Norfolk Floodwall.

5.7. STORM SURGE BARRIERS, TIDE GATES, AND FLAP GATES

Two types of gates were considered for storm surge barriers. They were sector gates (large openings, greater than 100 ft.), and miter gates (smaller openings, less than 100 ft.). Sluice gates (tide gates) were also considered for use in pump stations. 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.

5.7.1. 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 the mouth of the Miami River. Based upon the navigable channel width of 150, 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 20.9 ft. NAVD 88 and the bottom of the gate foundation at -18.6 ft. 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 5.12**) located in the New England District. This gate was selected as its geometry and opening width was similar to the one we anticipate for this 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 increased cost of construction. **Figure 5.13** shows a USACE New Orleans District Sector Gate in construction. This gate is the Bayou Dupre Sector Gate.

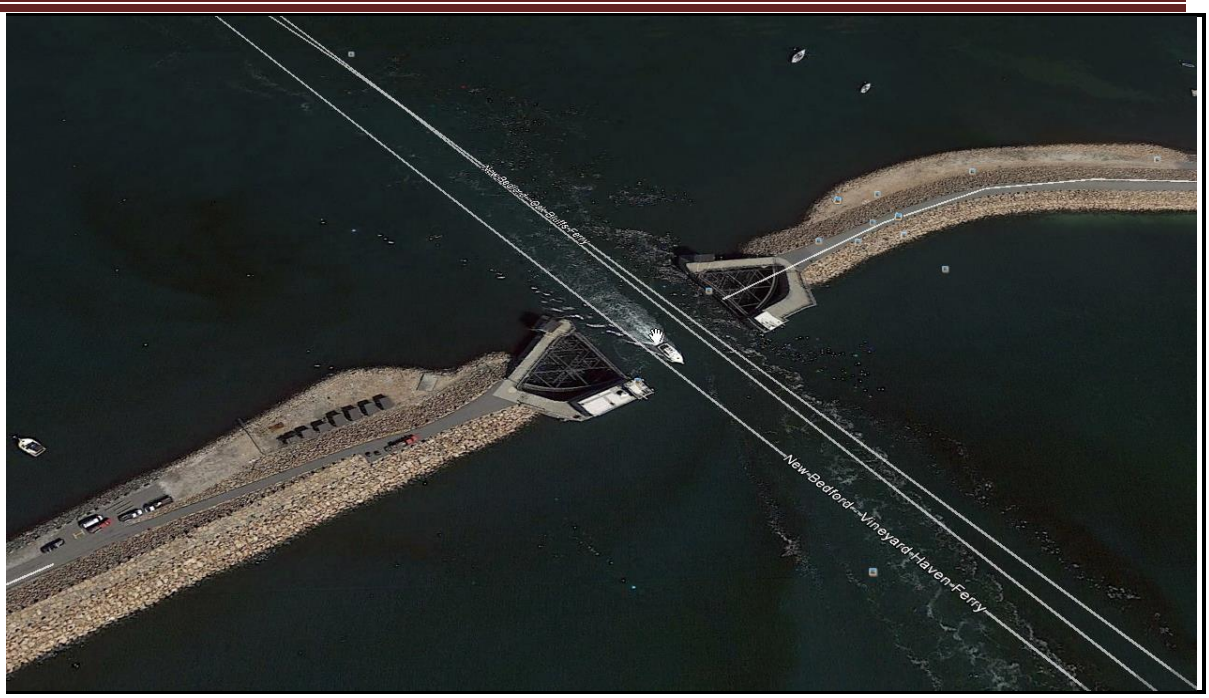


Figure 5.12. Bird's Eye View of the New Bedford – Fairhaven Barrier

Figure 5.13. Bayou Dupre Sector Gate under construction.

5.7.2. 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 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 5.14)** 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 average depths range between elevations -0.5 ft. NAVD 88 to -1.5 ft. NAVD 88. All of these dimensions work well with the miter gate designs we currently use. Miter gates also have relatively fast opening/closing times and when open, the gates recess along the abutment chambers for low space consumption. Cost estimates are based on existing gate constructions and operations, making adjustments to suit the Miami-Dade County CSRM study.

Figure 5.14. Image of Miter Gate at Richmond Filtration Plant flood Mitigation Project

5.7.3. SLUICE GATES

Tide gates or barriers that cross small tidal creeks, at drainage structures through a floodwall, or in pump stations 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 5.15** is an example of sluice gate through a levee.

Figure 5.15. Image of a Sluice Gate through a levee.

5.7.4. FLAP GATES

Flap gates (**Figure 5.16**) 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 5.16. Image of a Flap Gate at the end of a culvert.

5.8. 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. The 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 heighten the problem of interior flooding by blocking drainage outlets. During an interior drainage analysis, options of conveying the drainage to larger outlets will occur. There is a possibility of rerouting stormwater pipes to pass through the floodwall at specific locations or to a pump station. The stormwater pipes can been gravity driven or pressurized. Along the protected side length of the floodwall could be a stormwater pipe with grated inlets (for collection of rainfall runoff and/or rising groundwater) that would be directed to a pump station. Ideally, if pump stations are required to convey the runoff, the location would be near the river or canal outlets. Pump stations can be located away from the watershed outlet, but longer stormwater drain pipes will need to be provided for the outflow. Location of the outflow pipes will also be determined during the PED phase. Previous reports provided by Miami-Dade illustrated an analysis of some of the main watersheds including estimates of forward pump capacities. The large forward pump capacities

mentioned for the C-4 watershed will need to be considered in the design of the pump station for the lines of protection proposed. It is this large capacity that challenges the design and location of the Miami River watershed pump station location. A preliminary H&H analysis was done and compared the flooding extents to the CH2M: Surge and Flood Modeling for Miami-Dade County (Task 2.10) report dated June 2015 with similar flooding extents achieved. A more detailed interior flooding analysis will be evaluated in the PED phase of this study.

5.9. 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 HH&C, Geotechnical, and Structural Sub Appendices.

5.10. WAVE OVERTOPPING

Wave overtopping was analyzed using the SFLSSS water levels and wave heights. The overtopping analysis total probabilistic water levels included the USACE high curve SLR and storm surge. The overtopping analysis, for the designed vertical wall, used the interactive computer-based design and analysis system, ACES (Automated Coastal Engineering System), which is based on equations found in the U.S. Army 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 calculation of peak overtopping rates for the SFLSSS 5% Flood (20 year), 2% Flood (50 year), 1% Flood (100 year), and 0.5% Flood (200 year). ACES and EurOtop used both the Probabilistic and Deterministic approaches to analyze the overtopping for the design of the vertical wall.

5.11. QUANTITY ESTIMATES

Note: The section will be updated before the final Report. The following tables provide the quantities for each structural measure that was determined from the different project alignments. The tables also list which study areas (see **Figure 1.3**) that each structural measure resides in and the AEP SWL elevation (the SFLSSS water levels listed in **Section 5.2**) and height of floodwall of each structure. Quantities from the 5% Flood (20 year), 2% Flood (50 year), 1% Flood (100 year), and the 0.5% flood (200 year) project alignments are located in **Table 5.7** to **Table 5.10**.

Table 5.7. Miami River Project Alignment Quantities.

				South is defined as south of the Miami R				North is defined as north of the Miami R							
Option 1															
h5 pt 443	**Miami River**	southernmost alignment		South				North				South Wall in Water			
Frequency (years)	Annual Chance of Exceedance (ACE) in %	**2030** SWL (includes surge and SLR on a 90% Confidence Level)	**2079** SWL (includes surge and SLR on a 90% Confidence Level)	**2079** Top of wall elev (ft. NAVD88) incl Overtopping	Avg ground elev ft NAVD88 (south)	South Wall Avg Height incl overtopping (ft)	Length of South wall (ft)	**2079** Top of wall elev (ft. NAVD88) incl Overtopping	Avg ground elev ft NAVD88 (north)	North Wall Avg Height incl overtopping (ft)	Length of North Wall (ft)	**2079** Top of wall elev (ft. NAVD88) incl Overtopping	Avg ground elev ft NAVD88 (in water)- south	South Avg Wall Height in Water (ft)	Length of Wall in Water - South (ft)
20	5.00	4.8	7.66	7.9	7.4	0.5		9.5	5.5	4		13.2	-5.8	19	
50	2.00	6.52	9.38	10.9	7.4	3.5		13	5.5	7.5		16.2	-5.8	22	
100	1.00	7.74	10.6	13.4	7.4	6	3550	15.5	5.5	10	5840	18.2	-5.8	24	965
200	0.50	9.09	11.95	16.4	7.4	9		18.5	5.5	13		20.7	-5.8	26.5	
				min elev = 1.054		max elev = 13.415		min elev = -9.957		max elev = 9.817		min elev = -10.684		max elev = 0.977	

Surge Barrier				
2079 Top of wall elev (ft. NAVD88) incl Overtopping	Surge Barrier Avg ground elev (ft NAVD88)	SB Avg Wall Ht (ft)	Surge Barrier length (ft)	Total # of Closures (approximate)
15.9	-12.6	28.5		
18.9	-12.6	31.5		
20.9	-12.6	33.5	270	36
23.4	-12.6	36		
min elev = -16.205		max elev = -9.972		

Table 5.8. Edgewater Project Alignment Quantities (now deleted).

h5 pt 572	**Edgewater**			Wall				Wall in Water				
Frequency (years)	Annual Chance of Exceedance (ACE) in %	**2030** SWL (includes surge and SLR on a 90% Confidence Level)	**2079** SWL (includes surge and SLR on a 90% Confidence Level)	**2079** Top of wall elev (ft. NAVD88) incl Overtopping	Avg ground elev ft NAVD88	Wall Height incl overtopping (ft)	Length of wall (ft)	**2079** Top of wall elev (ft. NAVD88) incl Overtopping	Avg ground elev ft NAVD88	Wall Avg Height incl overtopping (ft)	Length of Wall (ft)	Total # of Closures (approximate)
20	5.00	4.29	7.15	9.1	5.1	4		11.4	-3.6	15		
50	2.00	5.57	8.43	11.6	5.1	6.5		13.9	-3.6	17.5		
100	1.00	6.58	9.44	13.6	5.1	8.5	6310	15.4	-3.6	19	7200	22
200	0.50	7.87	10.73	16.1	5.1	11		17.4	-3.6	21		
				min elev = 2.19		max elev = 12.304		min elev = -9.077		max elev = 1.957		

Table 5.9. Little River Project Alignment Quantities.

h5 pt 614	**Little River**			South				North				Surge Barrier				
Frequency (years)	Annual Chance of Exceedance (ACE) in %	**2030** SWL (includes surge and SLR on a 90% Confidence Level)	**2079** SWL (includes surge and SLR on a 90% Confidence Level)	**2079** Top of wall elev (ft. NAVD88) incl Overtopping	Avg ground elev ft NAVD88 (south)	South Wall Height incl overtopping (ft)	Length of South wall (ft)	**2079** Top of wall elev (ft. NAVD88) incl Overtopping	Avg ground elev ft NAVD88 (north)	North Wall Avg Height incl overtopping (ft)	Length of North Wall (ft)	**2079** Top of wall elev (ft. NAVD88) incl Overtopping	Surge Barrier Avg ground elev (ft NAVD88)	Surge Barrier Avg Wall ht (ft)	Surge Barrier length (ft)	Total # of Closures (approximate)
20	5.00	4.23	7.09	n/a	8.5	n/a		n/a	7.4	n/a		10.7	-0.3	11		
50	2.00	5.47	8.33	n/a	8.5	n/a		9.4	7.4	2		13.2	-0.3	13.5		
100	1.00	6.38	9.24	10	8.5	1.5	7380	10.9	7.4	3.5	4920	15.2	-0.3	15.5	85	85
200	0.50	7.31	10.17	12	8.5	3.5		12.9	7.4	5.5		17.7	-0.3	18		
				min elev = 3.297		max elev = 10.664		min elev = 1.125		max elev = 12.089		min elev = -1.558		max elev = 2.941		

Table 5.10. Biscayne Canal Project Alignment Quantities.

h5 pt 621	**Biscayne Canal**			South				North				Surge Barrier				
Frequency (years)	Annual Chance of Exceedance (ACE) in %	**2030** SWL (includes surge and SLR on a 90% Confidence Level)	2079 SWL (includes surge and SLR on a 90% Confidence Level)	**2079** Top of wall elev (ft. NAVD88) incl Overtopping	Avg ground elev ft NAVD88 (south)	South Wall Height incl overtopping (ft)	Length of South wall (ft)	**2079** Top of wall elev (ft. NAVD88) incl Overtopping	Avg ground elev ft NAVD88 (north)	North Wall Avg Height incl overtopping (ft)	Length of North Wall (ft)	**2079** Top of wall elev (ft. NAVD88) incl Overtopping	Surge Barrier Avg ground elev (ft NAVD88)	Surge Barrier Avg Wall ht (ft)	Surge Barrier length (ft)	Total # of Closures (approximate)
20	5.00	4.32	7.18	n/a	8.1	n/a		8.1	6.1	2		9.15	-1.35	10.5		
50	2.00	5.67	8.53	9.1	8.1	1		10.1	6.1	4		12.15	-1.35	13.5		
100	1.00	6.62	9.48	10.6	8.1	2.5	4600	11.6	6.1	5.5	4010	13.15	-1.35	14.5	95	41
200	0.50	7.55	10.41	12.6	8.1	4.5		13.1	6.1	7		14.65	-1.35	16		
				min elev = 3.016		max elev = 11.237		min elev = -3.039		max elev = 2.896		min elev = 2.779		max elev = 10.626		

5.12. COST ESTIMATES

The baseline cost estimate for the proposed measures and tentative selected plan were developed using MCACES in the Civil Works Work Breakdown Structure format. Quantities were calculated and provided by the designer engineers in the Huntington District. Quantities are shown in **Section 5.10** 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.

5.13. 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 SFLSSS water levels and the frequency wave heights. 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. To analyze risk of wave overtopping, different heights of the walls

were analyzed and adjusted 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 HH&C 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.

5.14. CONSTRUCTABILITY

The primary constructability issues for the Miami-Dade County Back Bay CSRM project are expected to be constructing adjacent to existing structures, karst limestone foundation, intruding into the Biscayne Bay, unknown soil contamination, space constraints in a highly urbanized area, and traffic impacts.

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.

The regional geology indicates the presence of karst limestone at some locations within the County of Miami-Dade. Shallow bedrock is also present and there is a constructability concern with driving sheet piling within these areas. Due to the local geology, design loads, congested working areas and rate of placement it is recommended that the T-Walls be supported by auger cast pile (ACP) foundations socketed into bedrock. See the Geotechnical Engineering Sub Appendix for more discussion on the ACP method.

Construction near and along the water will negatively impact road traffic and may impact boat traffic. The floodwall alignments of the Miami-Dade County Back Bay CSRM project cross numerous streets and parallel streets in some places. Boat piers along the shoreline in the Miami River area will also be challenging.

Resiliency can be defined as the ability of the 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 design of the floodwall being a T-wall will be adaptable in future height requirements and resilient in its impermeable construction. The pump stations will be adaptable and resilient in the design for having backup pumps included in the design and in its impermeable construction. 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 Miami-Dade County Back Bay CSRM project takes into consideration the effects of sea level rise 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 Hydrology, Hydraulics, 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 Miami-Dade County Back Bay CSRM, walls that are recommended to be 4 feet or higher, will be built as T-walls. As stated earlier in this report, all I-walls shall be constructed to a 6 feet height at a minimum; however, since we are required to consider adaptability, walls that are close to the 6 feet I-walls limit (preferably 4 feet minimum) will be constructed as T-walls to ensure that they can adapt. 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 procedures for the project and adaptation will be written in the OMRR&R Manual, which will be discussed more in **Section 5.15**. 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 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.

5.15. 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, Miami-Dade 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 floodwalls and other flood control structures that may be included in this study. 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 Miami-Dade County (the local sponsor).

USACE conducts two types of floodwall inspections: Routine Inspection and Periodic Inspection. Routine Inspection is a visual inspection to verify and rate floodwall system operation and maintenance. It is typically conducted each year for all floodwalls in the USACE Levee 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 Levee 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 floodwalls, interior drainage, pump stations, and channels. Each 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 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).

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 riprap and the areas adjacent to floodwalls, and from channels and waterways. 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. Pump stations also need to be completely overhauled periodically. Routine maintenance is expected in any project and can be planned for in advanced. The Cost Sub Appendix should be referenced for estimated Operation and Maintenance costs.

To assist with monitoring, certain tools and instruments are needed and measurements are required. Gages 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. 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 secant pile 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).

6.3. UPDATED SURVEYS

It is recommended that an updated topographic survey be performed during PED in areas of the City where new construction and road raising have occurred since 2018. New surveys may require an adjustment in the proposed height of walls, especially in areas where the road has been raised. 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.

6.4. 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), 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.

6.5. 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.

6.6. INTERIOR FLOODING ANALYSIS REFINEMENT

During the PED phase, Hydrology and Hydraulics (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 4.4 or later with the guidance of Engineering Manual 1110-2-1413 or approved USACE similar hydrologic model. 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 analysis may entail the use of the HEC-RAS (Hydrologic Engineering Center – River Analysis System) software version 5.0.7 or later in order to view the extents of the rainfall frequency in conjunction with tides.

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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14. USACE. 2018. Generation II Coastal Risk Model User's Manual. Army Corps of Engineers.

SUB APPENDICES

1. **STRUCTURAL SUB APPENDIX**
2. **GEOTECHNICAL SUB APPENDIX**
3. **HYDROLOGY, HYDRAULICS, & COASTAL SUB APPENDIX**
4. **COST ENGINEERING SUB APPENDIX**