US Army Corps of Engineers®

Norfolk District

Miami-Dade County Back Bay Coastal Storm Risk Management Study

Miami-Dade, Florida

HYDRAULICS, HYDROLOGY, & COASTAL SUB-APPENDIX

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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
G2CRM	Generation II Coastal Risk Model
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

This report presents the results of the Hydraulics, Hydrology and Coastal (HH&C) engineering evaluation and analysis for the Miami-Dade Coastal Storm Risk Management (CSRM) Study for the 10% engineering design. See **Figure 1.1** for image of the Miami-Dade County boundary.

All analysis discussed in this Draft Sub Appendix is subject to change as at the time of submission the preliminary analysis has not yet been completed.

The current tentatively selected plan (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.

As part of the Miami-Dade CSRM Study, the Hydrologic Areas were divided into four focus areas for the structural solutions. The focus areas are Miami River, Little River, Biscayne Canal, and Edgewater. Arch Creek is also discussed in this Sub Appendix for possible interior drainage relief. See **Figure 1.2** for the four individual focus areas within the county. Each hydrologic area represents one of the four focus areas in the County of Miami-Dade.

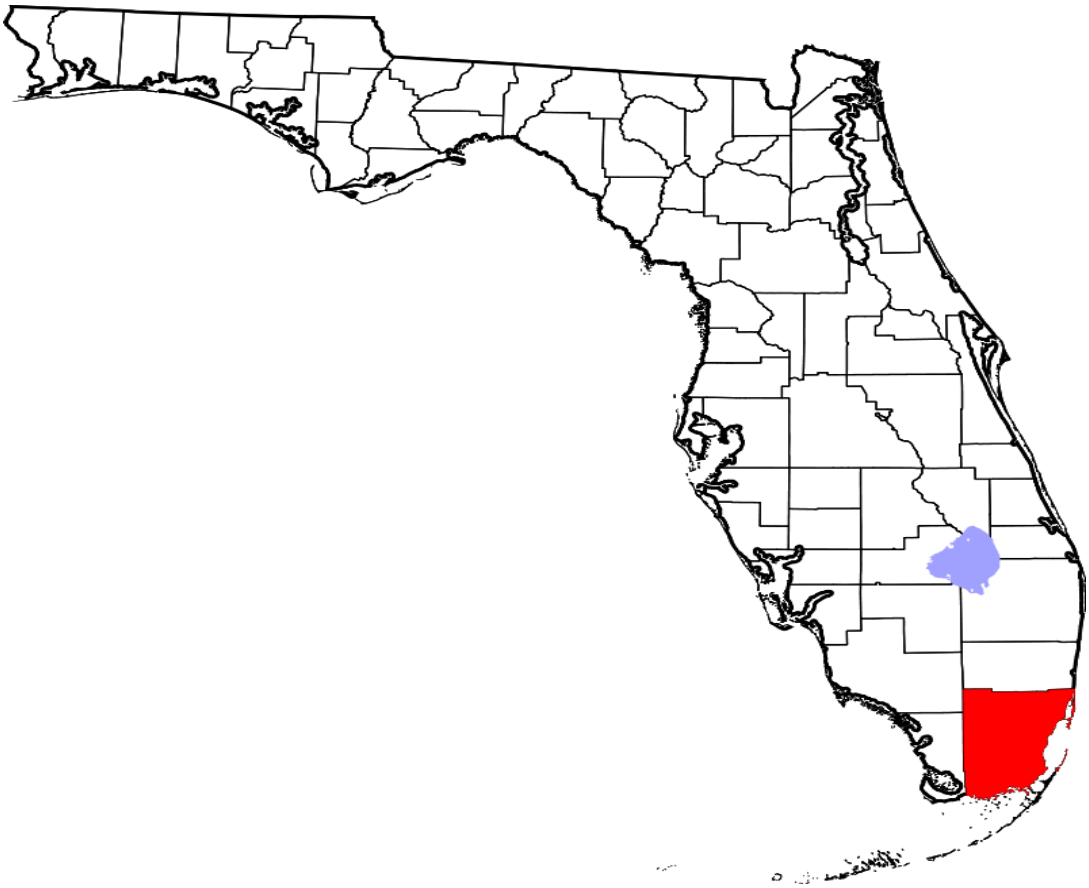


Figure 1.1. Location Map of Miami-Dade County (in red) within the State of Florida.

Miami-Dade County in Florida has experienced flooding from these types of coastal storms: tropical storms and hurricanes. The storms that impact the County are occurring more frequently. Another flooding event affecting Miami-Dade County are King Tides. King tides are caused by a combined gravitational pull between the sun and moon, when it is closest to earth, in the Northern Hemisphere. Near the end of a year higher tides and more water come ashore, according to the National Oceanic and Atmospheric Administration (NOAA). When the tides are exceptionally high, flooding in low-lying neighborhoods in coastal South Florida occurs. King tides occur annually between September and November in Miami and are lasting longer and extending farther inland. Some communities in Miami-Dade county prepare brochures of the predicted king tide schedules annually. When coastal storms coincide with king or high tides, the depth and extent of coastal flooding can increase dramatically. Weak winds that are blowing toward land during high-tide events can also

push water inland. Rainfall can also add a substantial volume of water to high-tide floods and coastal storm events.

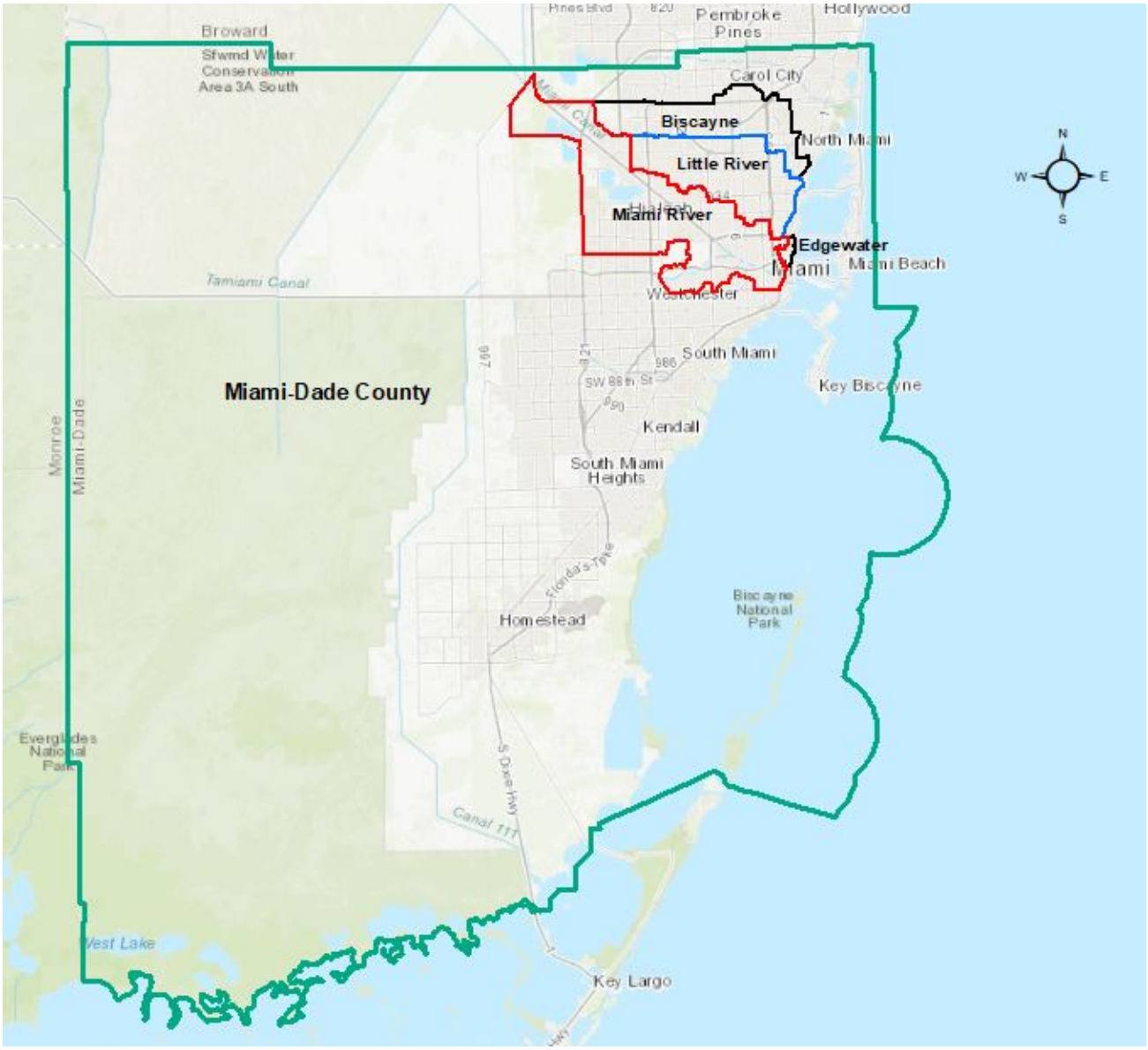


Figure 1.2. Miami-Dade County study extents and project locations.

Table 1.1 displays: (1) the date of historical storm events for Miami-Dade County, (2) the type of storm, (3) the peak water surface elevations where the water surface elevations reached over 0.0 feet North Atlantic Vertical Datum (NAVD88), and (4) the total rainfall from the event. The peak water surface elevations were measured by two NOAA tide gauges– Vaca Key, Florida Bay, FL tide gauge (#8723970) and Virginia Key, Biscayne Bay,

FL gauge (#8723214) referenced to NAVD88. The rainfall was measured at the Miami International Airport.

Many storms have occurred in Miami-Dade County according to the National Hurricane Center (NHC) Historic Storm Track database from this website: https://coast.noaa.gov/hurricanes/. See **Figure 1.3** for an image of the storm tracks. Not all storms in the image are listed in **Table 1.1**. The list of storms in **Table 1.1** includes hurricanes, high tidal events in the top ten list for the appropriate tide gauge, and events with measurable rainfall.

For this Miami-Dade County Back Bay CSRM the HH&C analysis examines the impacts and extents of coastal storm total water levels from the east (Biscayne Bay). Not covered in this study are effects of rising waters from the west side of the county which mainly encompasses rising water levels from the Everglades. Miami-Dade County experiences flooding from the west with the SFWMD structures controlling the water levels to allow pumping back to the west. There are many factors to consider that contribute to flooding in the Miami-Dade County. The complexity of the flooding issues has taken time to understand. In some areas of Miami the existing conditions since the start of this study have changed. New construction in the Miami area is subject to a new first floor elevation policy, one foot above the Base Flood Elevation required by FEMA.

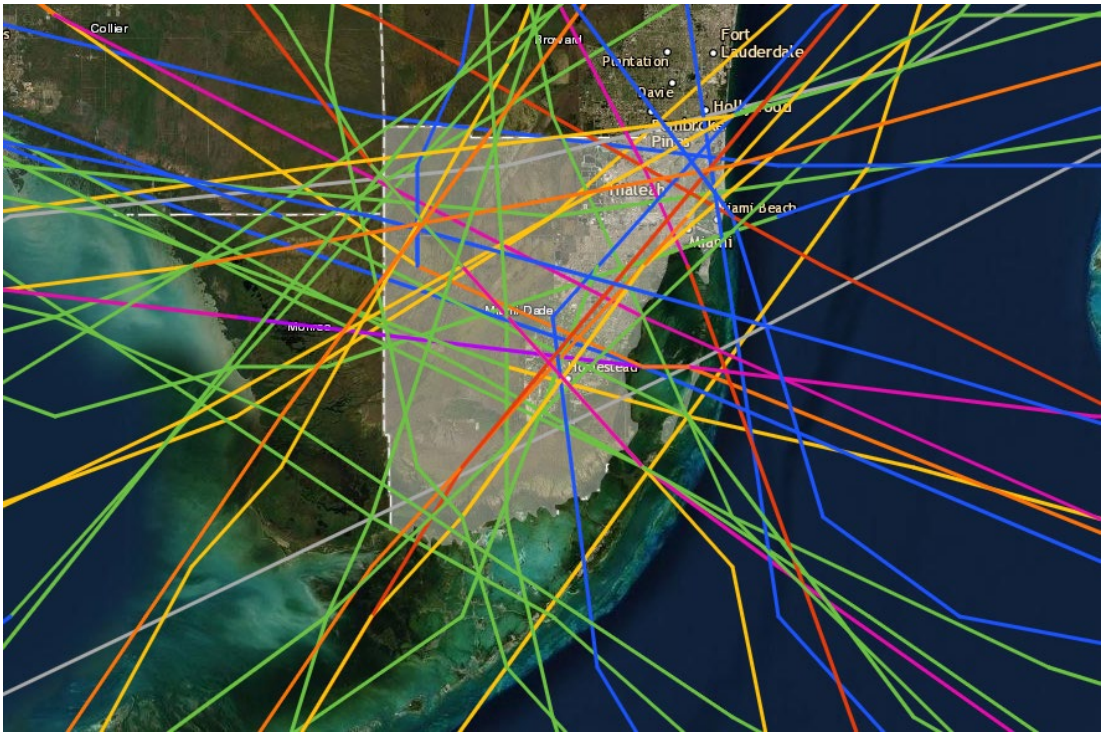


Figure 1.3. Storm Tracks from the NHC website for Miami-Dade County.

Table 1.1. Historical Storm Events in Miami-Dade County.

	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
15	September 21, 2004 (Ivan)	Extratopical	0.52 (0.96)	0.01
16	August 25-26, 2005 (Katrina)	Hurricane Cat 1	1.19 (1.19)	5.10
17	September 20, 2005 (Rita)	Hurricane Cat 1	0.66 (2.15)	0.81
18	October 24, 2005 (Wilma)	Hurricane Cat 2	5.44 (2.77)	0.76
17	July 23, 2010 (Bonnie)	Tropical Storm	0.12 (0.38)	1.5
18	September 10, 2017 (Irma)	Hurricane Cat 3	2.19 (3.79)	5.18
19	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

Chapter 3, Section 3.5 in the Main Engineering Appendix also discuss the historical storm events, water levels, and flooding that have impacted Miami-Dade County in Florida.

This report will discuss the existing information and reports that were reviewed and how that information was used in the HH&C engineering evaluation and analysis to come up with the contribution of the elements to get to the tentative selected plan for the study. The HH&C Sub Appendix will also provide recommendations for the next phase (Pre-Construction, Engineering and Design (PED) Phase) of the Miami-Dade County Back Bay CSRM study.

CHAPTER 2 South Florida Storm Surge Study

For the Miami-Dade County Back Bay CSRM, the Norfolk District used the South Florida Storm Surge Study (SFLSSS) data. The SFLSSS was for the Federal Emergency Management Agency (FEMA) and completed by a consultant in 2017. The SFLSSS intended to update the annual chance Stillwater level (SWL) elevations in the study area by applying storm surge modeling and statistical analysis. The United States Army Corps of Engineers (USACE) Engineering, Research and Development Center (ERDC) performed a statistical analysis on the SFLSSS 1992 raw water level data points for the project analysis and design. The South Atlantic Coast Study (SACS) was not completed at the time of the commencement of the Miami-Dade County Back Bay CSRM study. Once the SACS results are completed and available, the computed data used for this study can be compared for verification. The purpose of the SACS is to identify flood risk and plan and implement strategies to reduce the risk now and in the future, as well as, determine the magnitude and uncertainty of existing and future forcing conditions and concluded to use them to assess coastal engineering projects for coastal storm risk management and increase resiliency for the areas in the region. The SACS study focuses on the southeast coastal region of the United States (from North Carolina to Florida and west to Mississippi). See **Figure 2.1** for an image that depicts the SFLSSS study area.

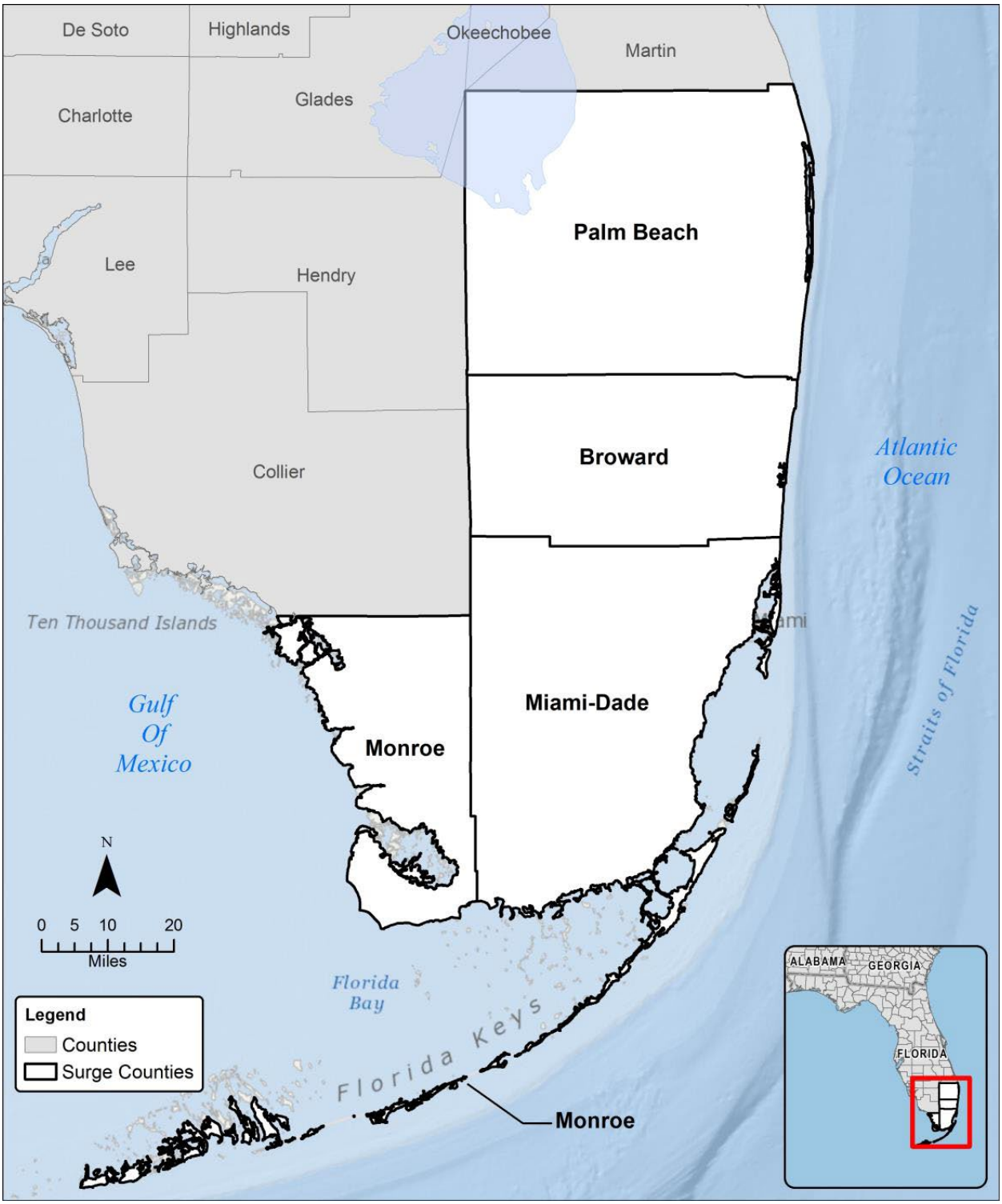


Figure 2.1 Image of SFLSSS area (image courtesy of BakerAECOM report).

The SFLSSS incorporated existing and future forcing and potential future climate change to perform statistical analyses and numerical hydrodynamic modeling for the region. The water level data points (save points) provided allowed ERDC to perform a statistical analysis on selected save points for the focus areas. The statistical analyses resulted in SWL elevations for the year 1992 as annual exceedance probabilities (AEP) for a 50% AEP (2 year flood), 20% AEP (5 year flood), 10% AEP (10 year flood), 5% AEP (20 year flood), 2% AEP (50 year flood), 1% AEP (100 year flood), 0.50% AEP (200 year flood), 0.20% AEP (500 year flood), and 0.10% AEP (1,000 year flood), for the 10%, 16%, 50% (mean), 84%, and 90% confidence limits. The higher the confidence limit (CL), the more reliable the elevation values become.

The numerical modeling study was performed using the two-dimensional hydrodynamic model ADCIRC and the two-dimensional spectral wave model SWAN. The ADCIRC model is a coastal circulation and storm surge model that uses the finite element method to solve the reformulated, depth-averaged shallow water equations. The model is run on a triangulated mesh with elevations derived from a seamless bathymetric/topographic DEM that includes both offshore and overland areas. The triangulated format of the mesh allows variation in the element size, so the study area can have a high concentration of nodes while fewer nodes (with higher element areas) can be placed farther away to make the mesh size more efficient without compromising accuracy (FEMA 2015). The SWAN model runs on the same triangulated mesh that is used with the ADCIRC model. During the model simulations, the water levels from ADCIRC are fed into the SWAN model at 15-minute interval (of model time). The SWAN model computes the wind-driven development of the storm waves, the propagation of the waves over the model domain, and the wave radiation stress gradients where the waves break close to the shore. In turn, the ADCIRC component is informed of the computed radiation stress gradients at the completion of each SWAN component time step. This information is used by the ADCIRC component to adjust the nearshore water levels for the wave-driven setdown and setup in the zone of breaking waves near the shoreline. This process continues for the duration of the wind and pressure forcing from the meteorological input files. The model was validated with historic tide gauge, high water mark, and wave buoy data.

The SFLSSS is documented in five Intermediate Data submittal (IDS) reports:

- IDS 1 is a detailed overview of the SFLSSS, including background on the study area, field reconnaissance, modeling mesh development, storm climatology, and the selection of the storm surge validation storms used to validate the model (FEMA, 2015). There are seven sections in the IDS 1 that discuss Technical Approach, Topo-Bathy DEM, Valid Storm Selection, Coastal Features, Climate, Winds and the HydroWave Model.

- IDS 2 describes the results of the tide and storm surge validation, mesh adjustments, development of the optimized set of synthetic hurricanes that fully represent the range of hurricane storm surge forcing that can occur in the project area based on an analysis of past events (the Joint Probability Modeling-Optimum Sampling [JPM-OS] storm set) (FEMA 2017).
- IDS 3 describe the development of computed maximum storm surge elevations for the series of synthetic hurricanes defines by the JPM-OS set (FEMA 2018).

- IDS 4 will describe the nearshore hydraulics, including applicable overland wave height, erosion, runup, and overtopping analysis.

- IDS 5 will include the draft work maps and information on the development of the study's product.

As this study is still ongoing and some files may not be shared until a formal review and appeals process has taken place, release was granted for the following files:

- Meshes
- Fort.13
- IDS Reports
- Validation Storm run files
- Production run Max Elevation files
- Production run sample fort.15/26 file

The unsorted modeling data was sent to ERDC for processing and statistical analysis as mentioned above.

2.1Datums

It should be noted that the native datum of the 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. Subsequently, all the data was converted from vertical datum Mean Sea Level (MSL) in meters to North American Vertical Datum 1988 (NAVD88). From the SFLSSS IDS 2 Section 2 report, "Additionally, tidal datums indicate more substantial variability in the relationships between the North American Vertical Datum of 1988 (NAVD88) and mean sea level (MSL) in the study area. The SFLSSS will follow the approach developed within the GANEFLSSS (Georgia and Northeast Florida) and ECCFLSSS (East Coast Central Florida) to account for the NAVD88 and MSL variation in the study area. A linear interpolation based on the water level offset at the northern and southern project extents will generate final water levels referenced to NAVD88 and account for the non-uniform MSL-NAVD88 offset."

Unless otherwise noted, all SWL elevations stated in this report are referenced to the North American Vertical Datum 1988 (NAVD88). For the Hydraulics and Hydrology (H&H) analysis the Virginia Key tide gauge is physically located close to the proposed structural solutions and the peak tide data was used produce inundation maps. **Figure 2.2** displays the tidal datum for the Virginia Key gage relative to NAVD88**.**

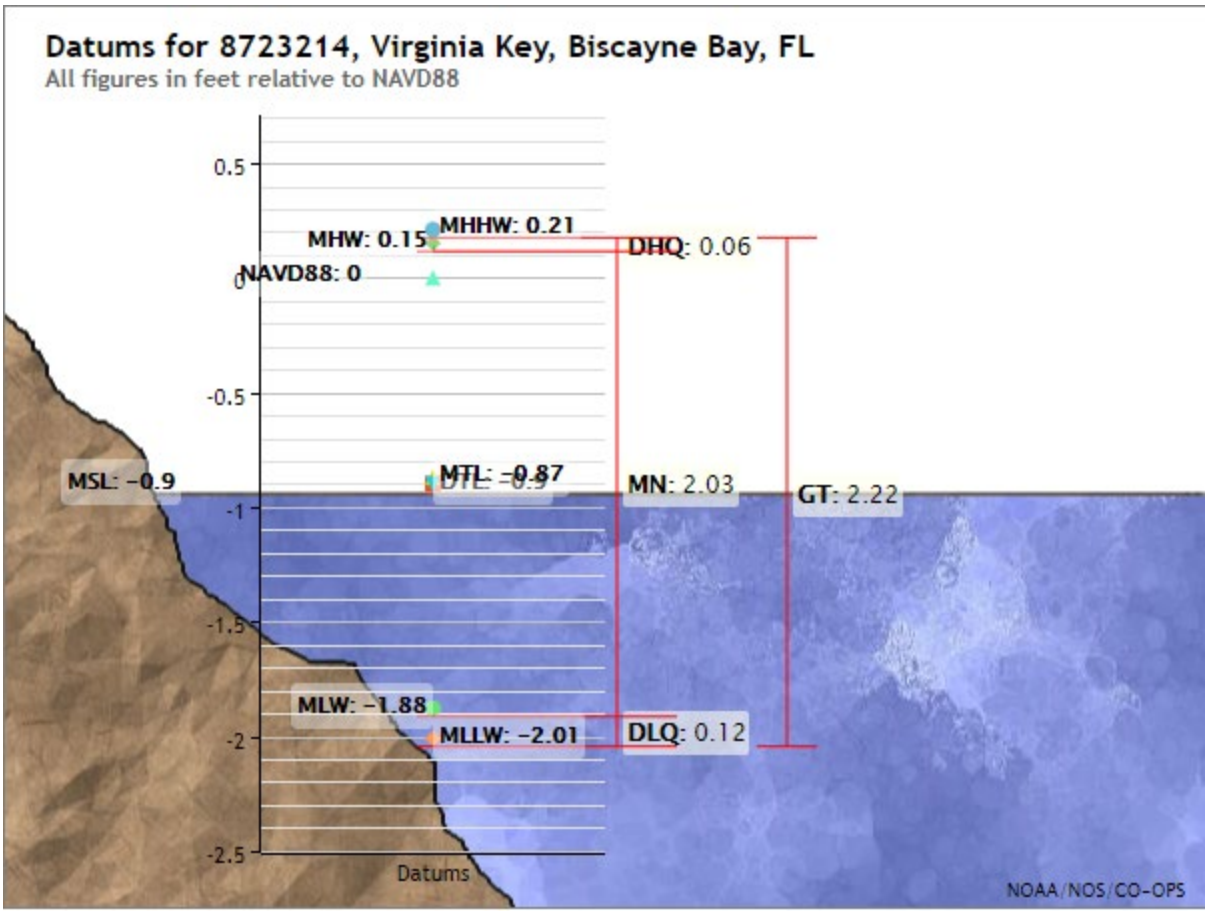


Figure 2.2. Virginia Key, Florida Bay, FL datum diagram (courtesy of NOAA).

2.2Digital Elevation Model (Dem) Development

The following discussion references the document titled "Technical Approach IDS1:Section 1, Task Order 99 – South Florida Flood Insurance Study, Version 2.0, September 2014" that was provided as documentation.

A seamless topo-bathy Digital Elevation Model (DEM) included Light Detection and Ranging (LIDAR) and hydrographic survey points. "The major source for the topographic data is regional LIDAR data. Most of the bathymetric data comprises a combination of data from National Ocean Service (NOS), U.S. Army Corps of Engineers (USACE), and the South Florida Water Management District (SFWMD). Some of the bathymetric data, such as the National Ocean Service (NOS) data, will require vertical transformation to a common datum, NAVD88, using the National Oceanic and Atmospheric Administration's (NOAA) VDatum tool (http://vdatum.noaa.gov/). Both topographic and bathymetric datasets may

require re-projection or transformation into a uniform vertical and horizontal datum (i.e., NAVD88 and Universal Transvers Mercator, UTM, respectively) before being merged into the final seamless DEM. Notably, the DEM development will occur in concert with the ADCIRC mesh development. This parallel effort ensures that interpolation of the DEM elevations onto the model mesh produces high resolution of features known to alter surge propagation in the coastal zone. To facilitate file transfer and application in the SWAN+ADCIRC model, the final seamless DEM deliverable will be provided by tile indexes covering the study region."

The topographic data included several LiDAR sources noted in the following figures from the IDS 1 Section 2 Report:

Table 2. Inventory of Digital Topographic Data

Year	Description	Data Type	Source/Owner
2009	TopoBathy of East Coast Shoreline	Airborne LIDAR	NOAA
2007	St. Lucie and Martin Counties, FL LIDAR	Airborne LIDAR	FDEM
2007	Palm Beach County, FL LIDAR	Airborne LIDAR	FDEM
2007	Herbert Hoover Dike Project, FL LIDAR	Airborne LIDAR	FDEM
2000	Palm Beach County, FL 1 ft contours	Contours derived from Airborne LIDAR	USACE
2001	Palm Beach County, FL LIDAR	Airborne LIDAR	FIU
2001	Palm Beach County, FL LIDAR (DEM)	Airborne LIDAR	SFWMD
2007	Broward County, FL LIDAR	Airborne LIDAR	FDEM
2007	MiamiDade County, FL LIDAR	Airborne LIDAR	FDEM
2008	Florida Keys Project, FL LIDAR	Airborne LIDAR	FDEM
2007	Monroe County, FL LIDAR	Airborne LIDAR	FDEM
2007	Collier County, FL LIDAR	Airborne LIDAR	FDEM
2010	TopoBathy of West Coast Shoreline	Airborne LIDAR	NOAA
Various	USGS National Elevation Data (10 meter DEMs)	Digital Elevation Model	USGS
2014	South FL Composite Topography	Digital Elevation Model	SFWMD
Various	Beach Profiles – Multiple Counties	Survey	Multiple Counties

Table 3. Base Topographic Data Used

Year	Description	Data Type	Source/Owner
2007	St. Lucie and Martin Counties, FL LIDAR	Airborne LIDAR	FDEM
2007	Palm Beach County, FL LIDAR	Airborne LIDAR	FDEM
2007	Herbert Hoover Dike Project, FL LIDAR	Airborne LIDAR	FDEM
2001	Palm Beach County, FL LIDAR (DEM)	Airborne LIDAR	SFWMD
2007	Broward County, FL LIDAR	Airborne LIDAR	FDEM
2007	MiamiDade County, FL LIDAR	Airborne LIDAR	FDEM
2008	Florida Keys Project, FL LIDAR	Airborne LIDAR	FDEM
2007	Monroe County, FL LIDAR	Airborne LIDAR	FDEM
2007	Collier County, FL LIDAR	Airborne LIDAR	FDEM
Various	USGS National Elevation Data (10 meter DEMs)	Digital Elevation Model	USGS
2014	South FL Composite Topography	Digital Elevation Model	SFWMD

Bathymetric datasets were collected from 17 different organizations. The complete listing of datasets which were located, obtained, and reviewed for inclusion in the bathymetric DEM have ten tables (See Appendix B in the IDS1 Section 2 report) of sources from the following agencies: NOS, USACE, Jacksonville District (channel surveys), USACE-Joint Airborne LiDAR Bathymetry Technical Center of Expertise (JALBTCX), Palm Beach County, Florida Inland Navigation District (FIND), National Aeronautics and Space Administration (NASA), SFWMD, USGS, and others.

The final combined topographic and bathymetric surface was exported to a DEM with a cell resolution of 15 feet for interpolation to the ADCIRC mesh.

The point elevation data from the SFLSSS was used in the Environmental Systems Research Institute (ESRI) Geographic Information System (GIS) software, ArcMap. The inverse distance weighted tool was used to produce the raster DEM with a 5 foot resolution. Bathymetry in the Biscayne Bay and in most of the coastal canals is included. An interpolated DEM of Miami-Dade County from the FEMA SFLSSS is shown in **Figure 2.3.**

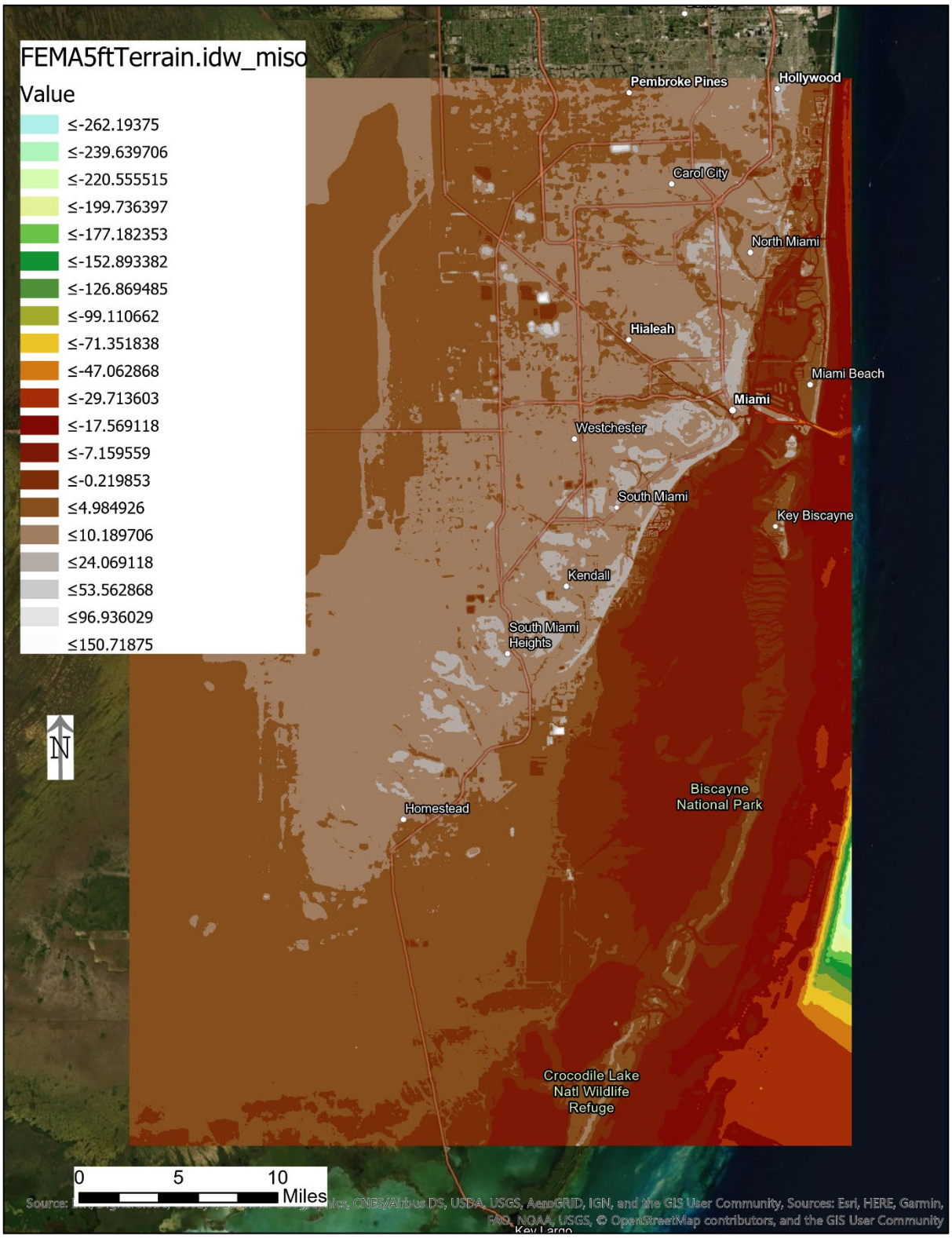


Figure 2.3. Miami-Dade County DEM

2.3SFLSSS Storm Selection and Statistical Method Development

The following discussion references the document titled "Validation Storm Selection IDS1: Section 3, Task Order 99 – South Florida Flood Insurance Study, Version 4.0, February 2015: Final Report" that was provided as documentation with the SFLSSS data.

Within the SFLSSS project area, major events that contribute to the AEPs for 2% (50 year), 1% (100 year), 0.5% (200 year), 0.2% (500 year), and 0.1% (1000 year) are rare and the spacing of tide gauges is large compared to the characteristic scale of differenced in maximum hurricane storm surge levels, the available tide is not sufficient to determine these flood levels throughout the project area. Instead, numerical modeling of the storms and their resulting surge heights are relied upon to represent the maximum coastal flood elevations at points across the SFLSSS project area. For more frequent events, 50% (2 year), 20% (5 year), 10% (10 year), and 4% (25 year) events, the tide gauge data is sufficient to develop the Stillwater elevations as the gauge record lengths are much longer than the return periods themselves.

To develop the SFLSSS 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 Southeast Florida coast. Extra-Tropical storms were not included in the SFLSSS Study because previous FEMA studies in Florida provided evidence that they do not contribute to AEPs for the 2% (50 year), 1% (100 year), 0.5% (200 year), 0.2% (500 year), and 0.1% (1000 year) storm surge heights.

The SFLSSS Study followed the Joint-Probability Method (JPM) as described by Resio (2007) and Toro et al (2010a) and incorporated experience from past FEMA flood studies along the Southeast Florida Coast. Appropriate data and model modifications captured the conditions in Southeast Florida. The JPM method considers all possible combinations of

storm characteristics as landfall, with their associated probabilities, calculates the surge effects for each combination, and then combines these results to obtain the annual probability of exceeding any storm surge elevation of interest. This calculation is represented as a multi-dimensional integral (the JPM integral). The SFLSSS Study used an approach referred to as the JPM-OS, where the OS refers to "optimum sampling", based on a procedure that approximates the multi-dimensional JPM integral by means of a weighted sum over a manageable number of discrete probability masses (Toro et al. 2010b). Each of these masses may be interpreted as the characteristics of a representative synthetic storm at landfall. These characteristics, together with some simple deterministic rules, are used to specify the entire storm history, which is then used as the as input to the numerical wind, wave, and surge models. The JPM-OS approach developed a representative set of 392 synthetic storms and their associated annual recurrence rates. These storms and their rates provide a condensed representation of the population of possible future synthetic storms used to calculate surge inundation probabilities

2.4Miami-Dade County CSRM Storm Database

A numerical hydrodynamic model (ADCIRC), forced by the synthetic storms, is used to translate the probabilistic model of storm conditions to the corresponding probabilistic definition of annual exceedance probabilities (AEP) for coastal floods. FEMA provided the raw, unsorted modeling data for processing. While the documentation for the SFLSSS stated 392 storms were run for the production runs, the modeling data provided to ERDC included the responses for 390 storms. Additionally, some of these files were corrupted. Ultimately ERDC processed the data for 390 storms. The statistical analysis performed by ERDC on the save points produced maximum year 1992 Stillwater Level (SWL) AEP curves and maximum wave AEP curves (heights and periods). SWL by USACE definition has 3 main components: mean sea level, astronomical tide, and storm surge. Also, as part of the hydrodynamic modeling, wave set-up is also included (SWL + wave set-up is sometimes called Dynamic SWL, or DSWL).

ERDC produced statistics and times series results for 779 selected save points around Miami-Dade County. See **Figure 2.4** for some of the save points shown that were selected.

ERDC's methodology to calculate the AEPs was the same joint probability analysis performed for SFLSSS, but how the final results are presented differ. For example, for SFLSSS only one hazard curve is produced for each point, whereas for the USACE, a family of five (5) curves is generated to characterizes and convey the uncertainty; this is a mean curve (50%) and 4 non-exceedance confidence limits, 10%, 16%, 84%, and 90% for the Miami-Dade County storm statistics. Save points near proposed structural measures were reviewed and selected where the SWLs, wave heights, and time periods had data for the four AEPs chosen.

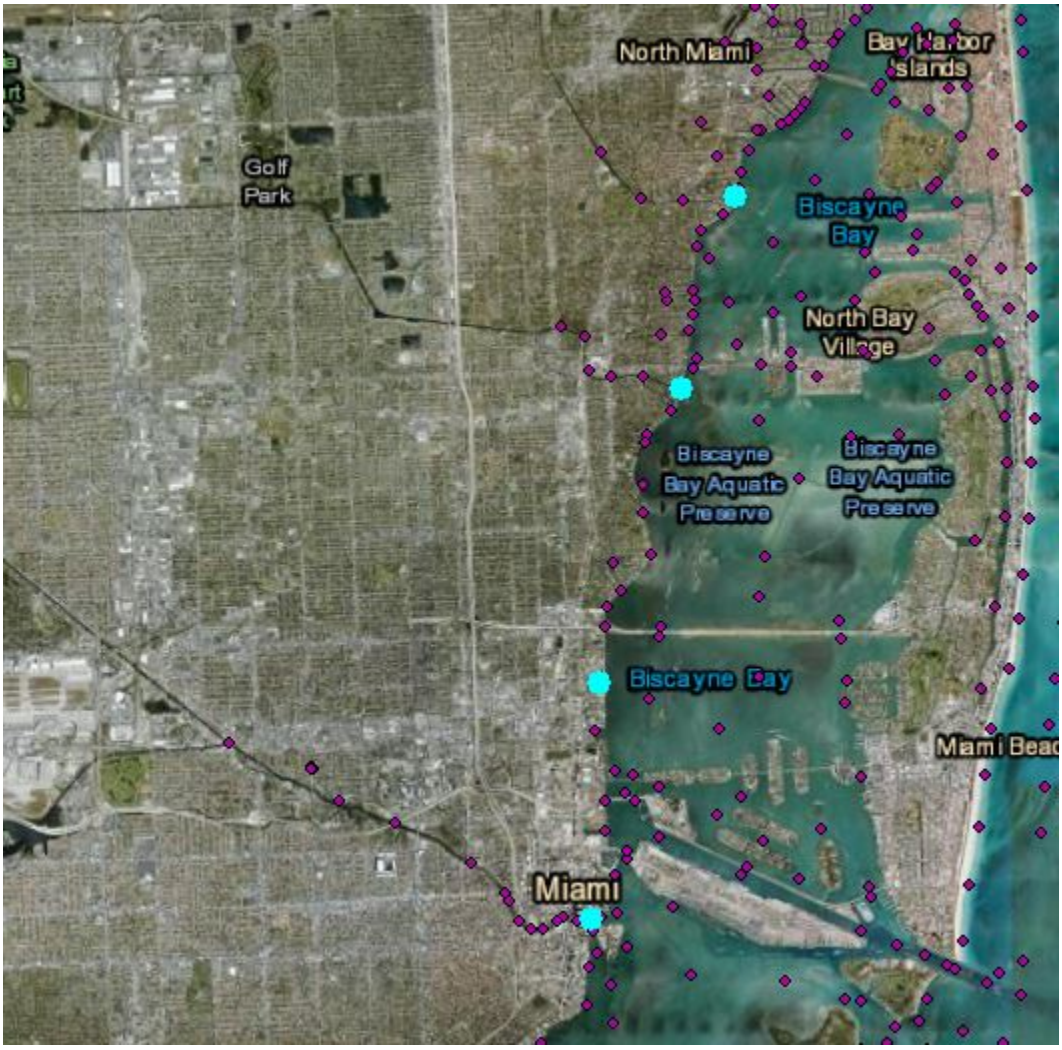


Figure 2.4 SFLSSS model save points for Miami-Dade County (the purple circles in the image – the blue nodes are the saved points used for the four focus areas).

2.5Confidence Limits and Risk Discussion

As previously stated, the ERDC results computed and provided statistical water levels, wave heights, and wave periods for different storm frequencies and confidence limits. **Figure 2.5** to **Figure 2.8** displays Stage-Frequency Curves from the ERDC results at save points in each Miami-Dade County Back Bay CSRM study area for the mean (50%), 10%, 16%, 84%, and 90% confidence limits for water levels.

The following figures include the adjusted 1992 values to the year 2018. The adjustment was based on the USACE low curve for Sea Level Rise from 1992 to 2018 to represent what has occurred. The sea level rise value is 0.31 feet and is based on the Vaca Key, Florida Bay, FL NOAA tide gauge #8723970.

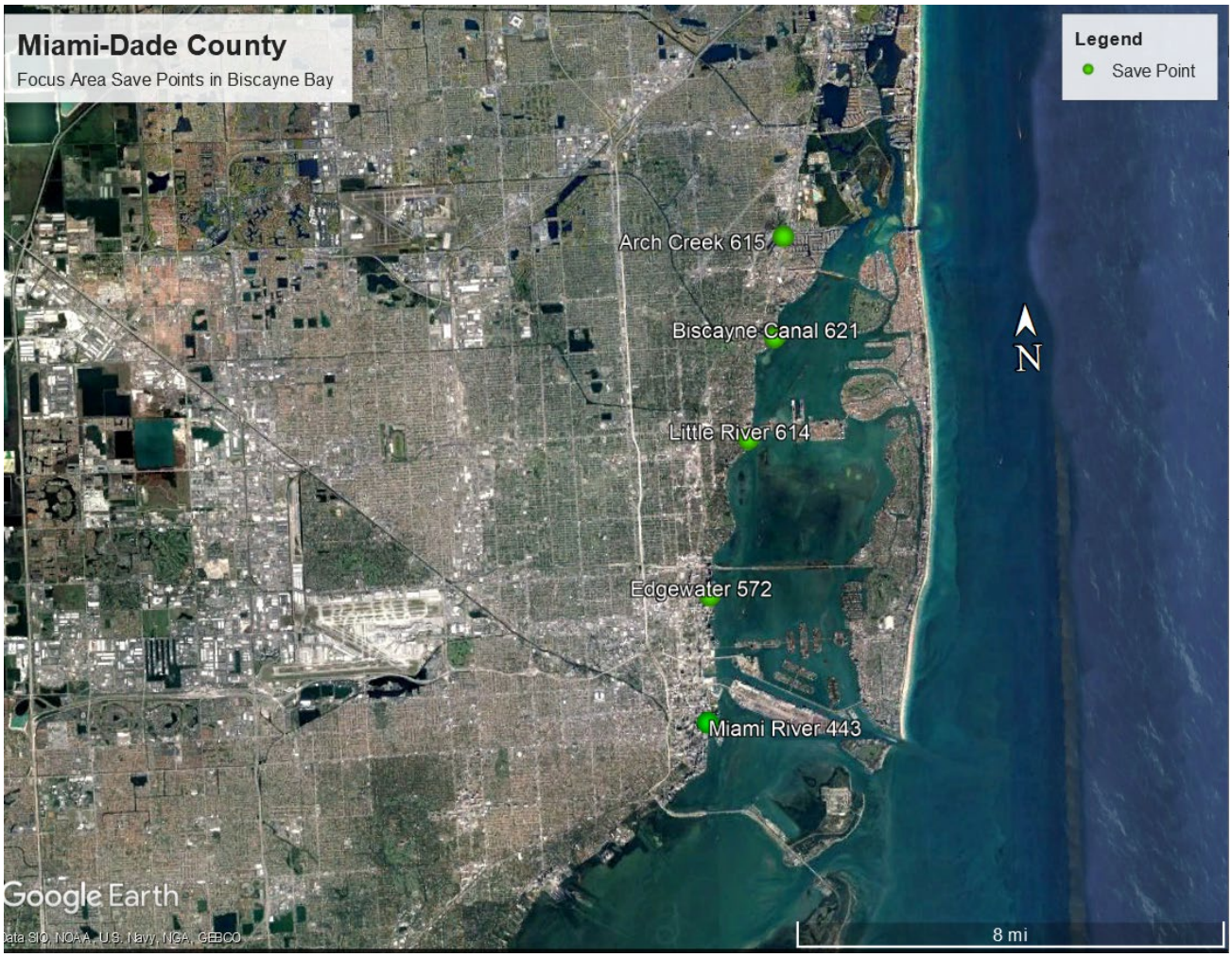


Figure 2.5. Locations of Stage-Frequency Curve save points in Figure 2.6 to 2.10.

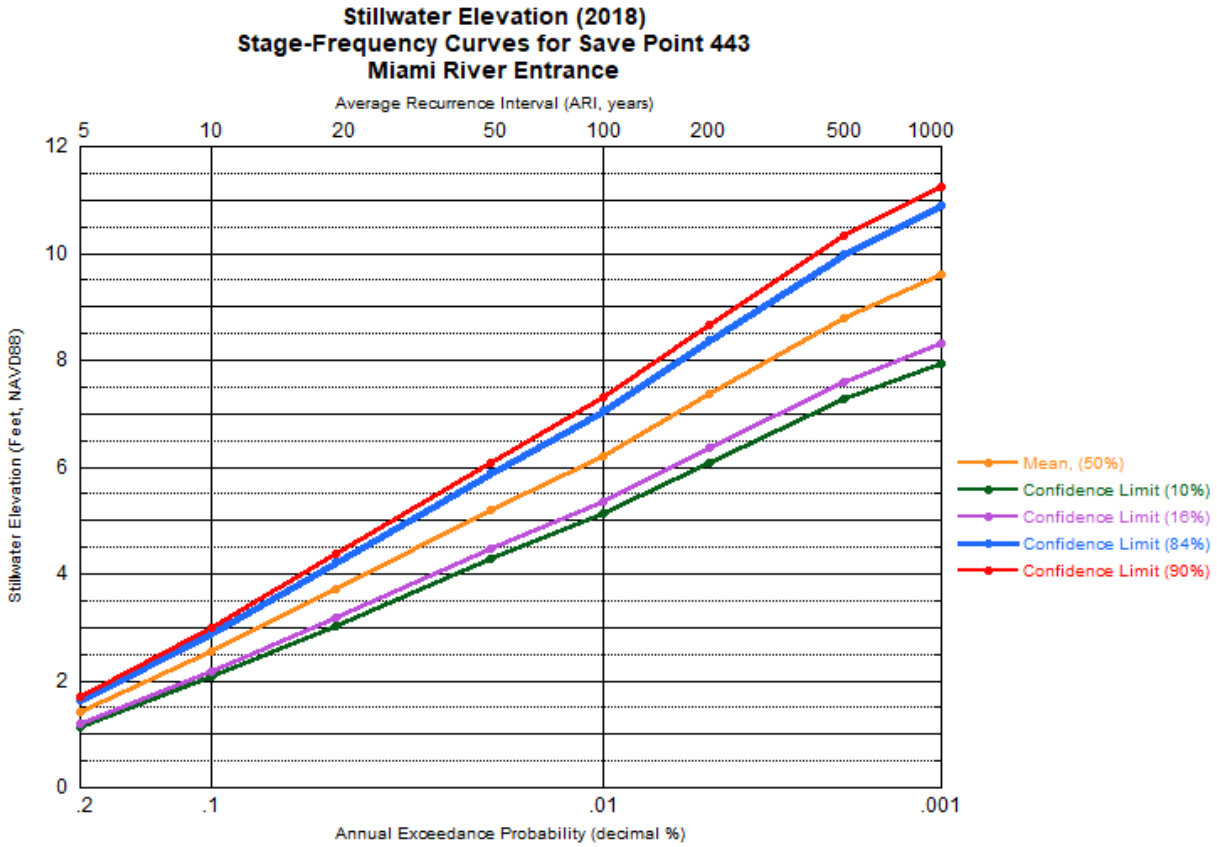


Figure 2.5. Stage-Frequency Curve for Save Point 443 (confidence limits shown).

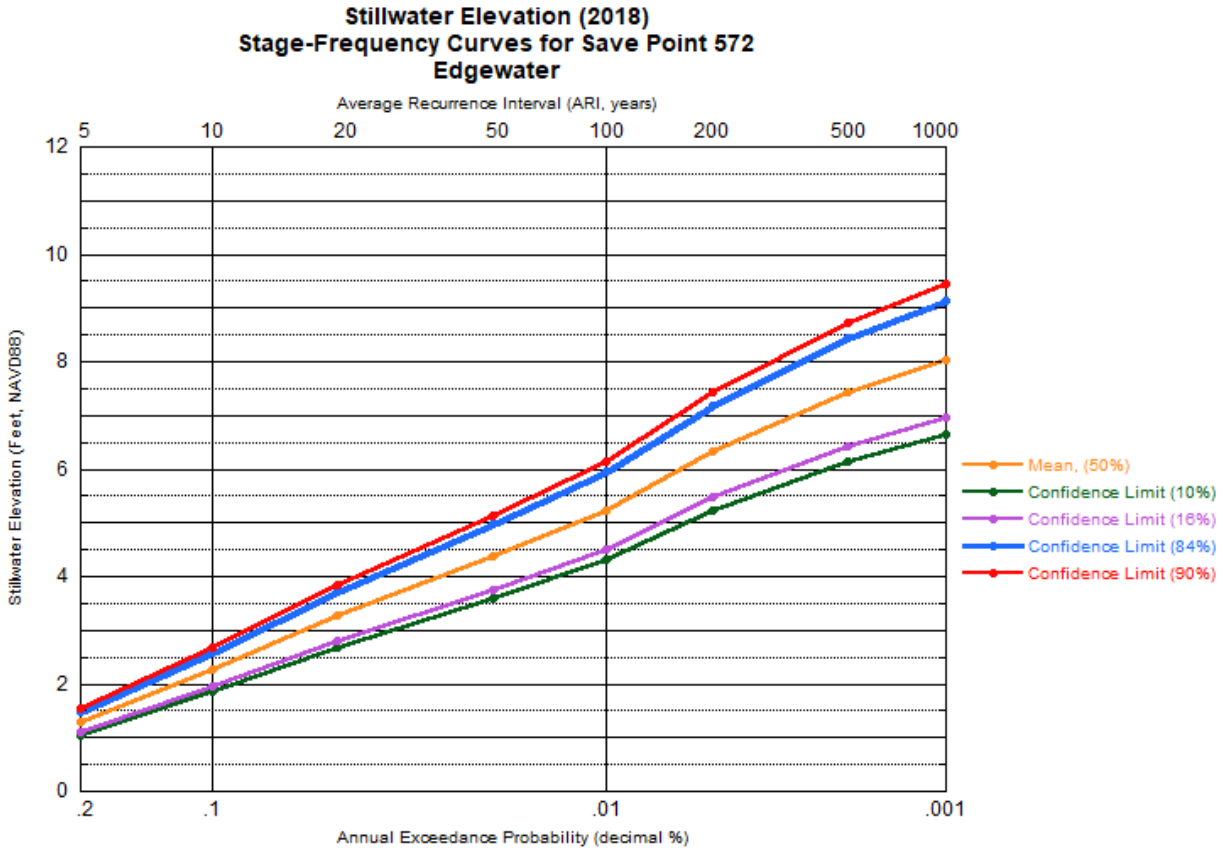


Figure 2.6. Stage-Frequency Curve for Save Point 572 (confidence limits shown).

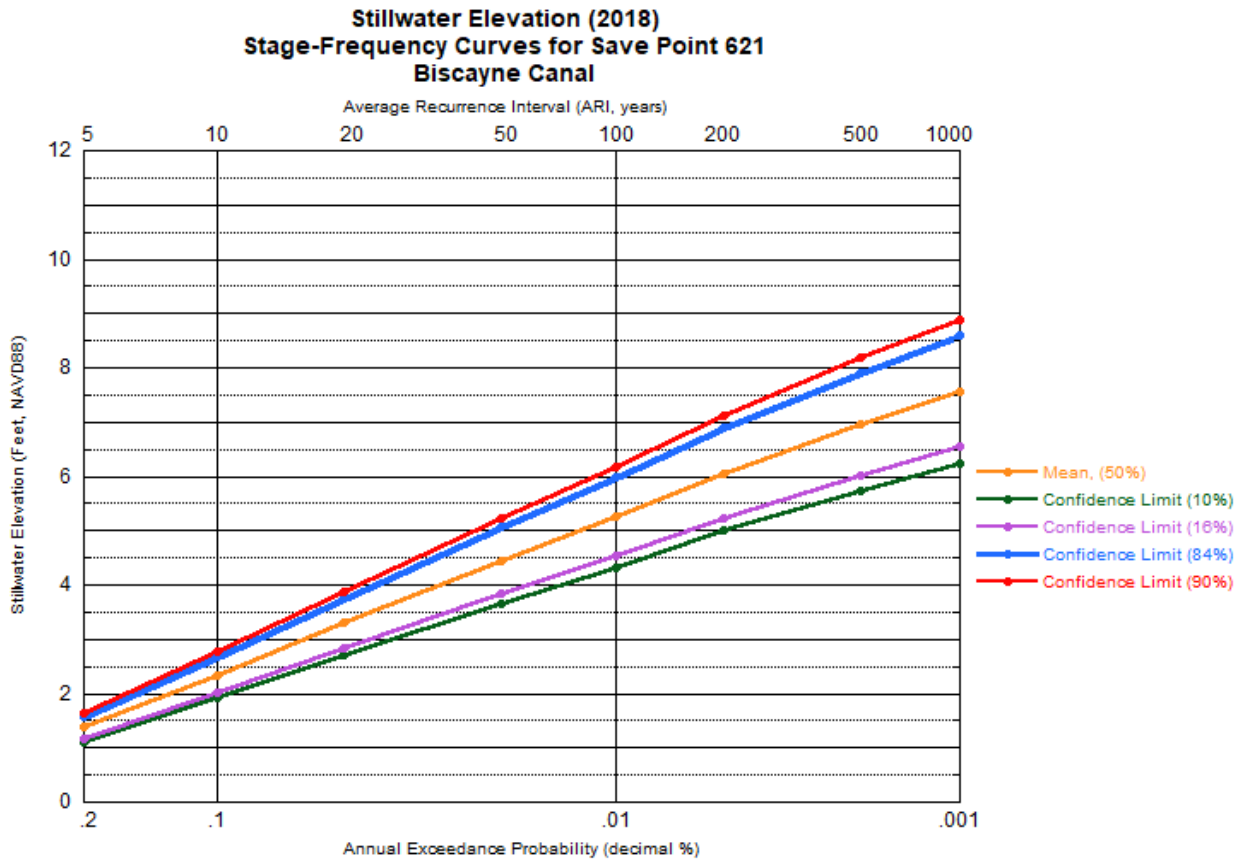


Figure 2.7. Stage-Frequency Curve for Save Point 621 (confidence limits shown).

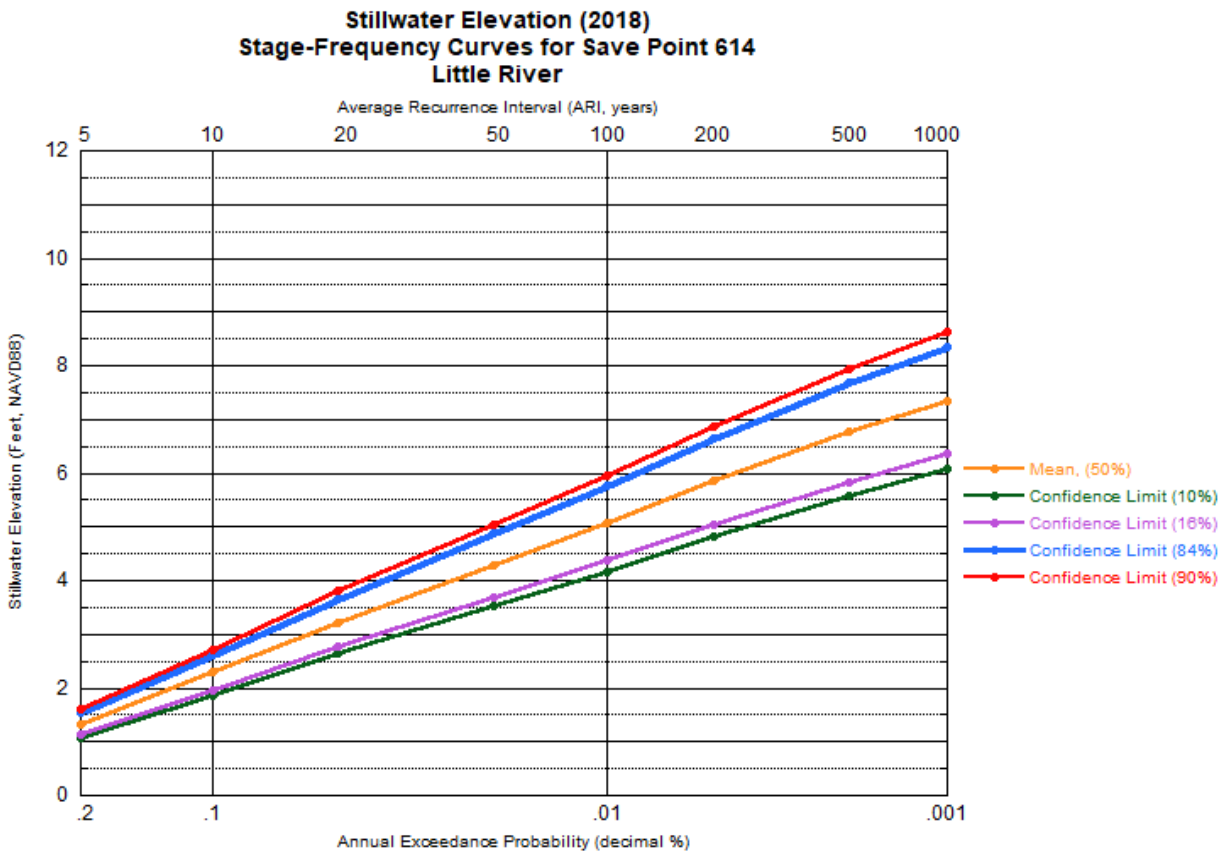


Figure 2.8. Stage-Frequency Curve for Save Point 614 (confidence limits shown).

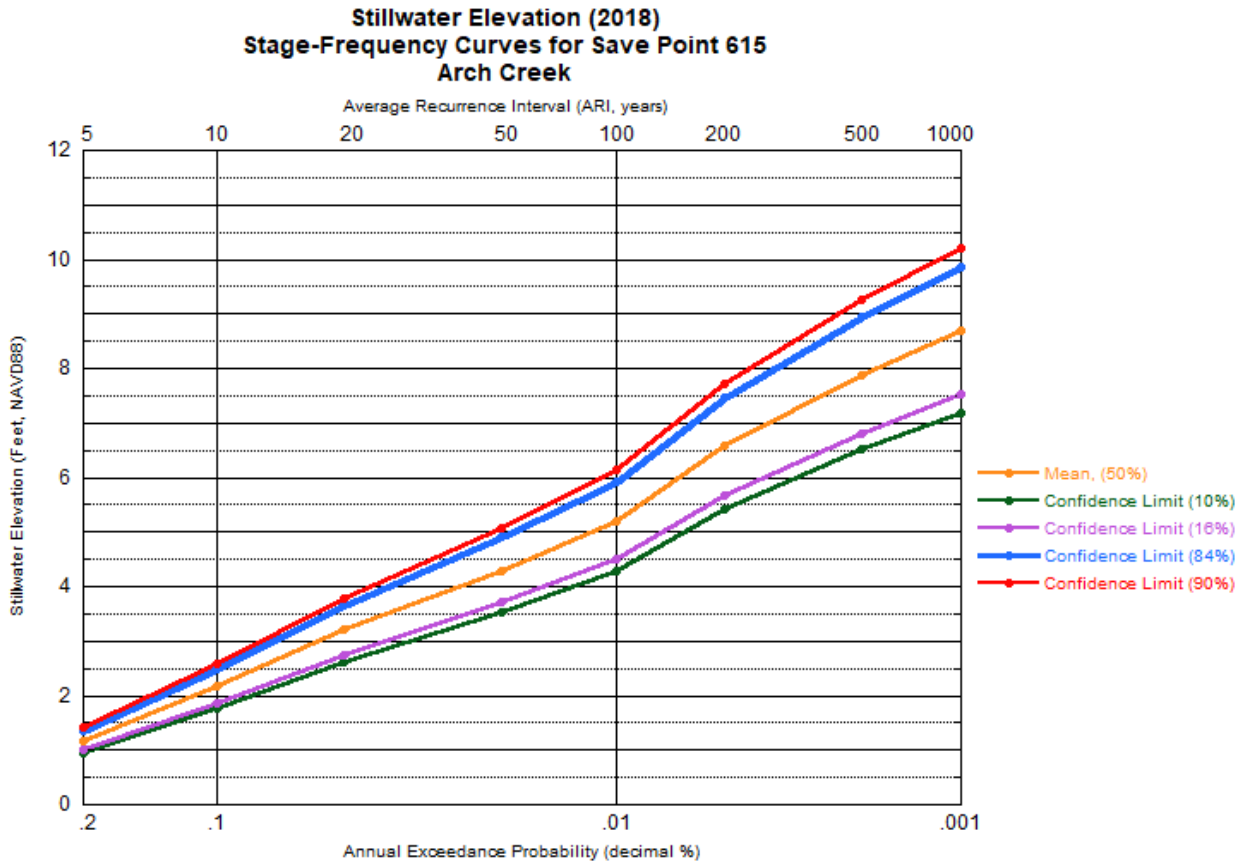


Figure 2.10. Stage-Frequency Curve for Save Point 615 (confidence limits shown).

Risk-based discussion involves comparing the 50%, 84% and 90% confidence limits and how that impacts adaptive management and if the confidence limits impact wave overtopping. A design event corresponding to a particular frequency of storm event with an appropriate level of assurance of being contained by the authorized alternative, is typically determined through the planning process and establishes the design floodwall and levee grade. It is generally desired to choose a higher confidence limit (i.e., 84% or higher). If deemed appropriate, additional superiority may be added to the design grade (top of a floodwall). The top-of-floodwall design level is based on the elevation which limits the overtopping rates to some appropriate allowable rate. For the Miami-Dade County Back Bay CSRM, the 90% Confidence Limit (CL) was chosen to formulate alternatives which led to the tentatively selected plan. **Figure 2.9** depicts a risk based concept for a floodwall structure.

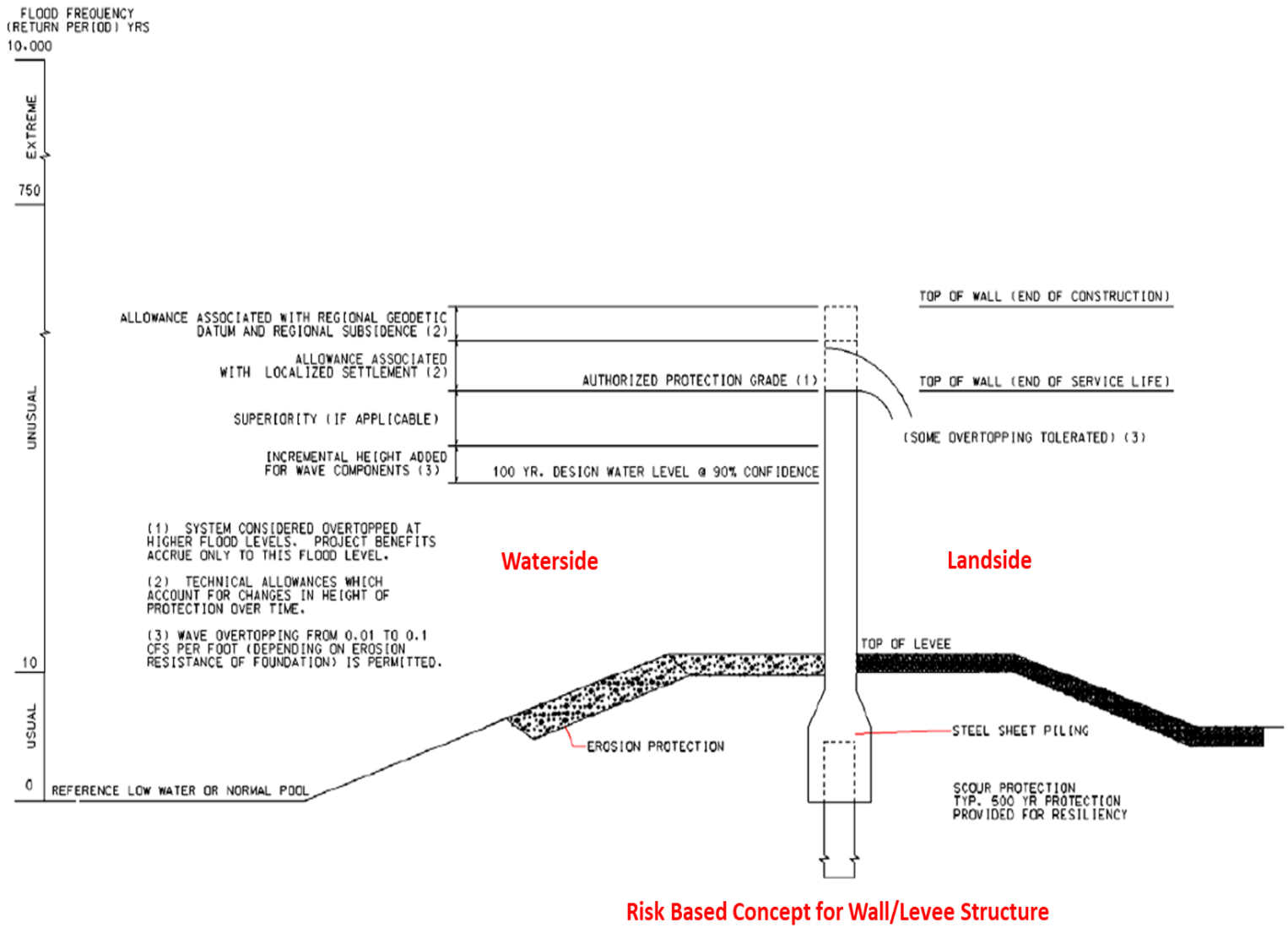


Figure 2.9. Image modified from EC 1110-2-6066.

The 90% CL SWL elevations depending on the AEP are approximately 0.25 feet to 1.66 feet greater than the 50% CL for the five save points mentioned above. The 90% CL SWL elevations are approximately 0.89 feet to 1.09 feet greater than the 50% CL for the AEP 1% (100 year) SWL elevations and were chosen for initial the design water levels for the floodwalls.

During the PED phase, higher resolution and locally calibrated models will be used to model these complex areas, and water levels will be refined. As mentioned before the results of the SACS will also influence the water levels. The ultimate design will rely upon a confidence interval chosen in consideration of all the risks and consequences facing the project, in accordance with ER 1105-2-101.

The current tentatively selected plan (TSP) proposes T-walls, which would allow the height of the walls to be adaptable to higher water surface elevations. In this case higher SWL elevations from higher frequencies (greater than 100 years) would require a higher wall to be constructed in the focus areas in Miami-Dade County and additional wall length may also need to be added. ERDC also provided AEPs for different CLs for the SWL elevations, for wave heights, and for wave time periods. Therefore, since the TSP design used the 100 year frequency wave heights to adjust the heights of the wall to account for the allowable wave overtopping, and should the SWL elevations increase or decrease, the heights would be adjusted to account for the change in the SWL elevation.

It is recommended that the array of SWL AEPs discussed be evaluated by the economics team to better determine the design water level yielding the greatest net benefits.

2.6Historical Tide Gauge Analysis vs. Computer Modeling

There have been discussions in the past about computing frequency water levels from a historical tide gauge analysis versus computer modeling (in this case the SFLSSS ADCIRC modeling). The historical record at the NOAA Vaca Key, Florida Bay, FL gauge (#8723970), established in 1970, does not include most of the historical storms listed in **Table 1.1**. The

historical maximum water levels are approximately equal to a 99% AEP flood to 10% AEP flood (1 to 10-year) tidal events. The extreme water level events ranged from a 10% AEP to a less than 1 % AEP. A statistical gauge analysis of the historical record may suggest that what has occurred in the past will occur in the future, this may underestimate the risk. Modeling effects, such as what was done for the SFLSSS, provide an opportunity to evaluate impacts of stronger hypothetical storms that may not have occurred on record, but could occur.

CHAPTER 3 Impacts of Climate Change

3.1Guidance

Climate change is defined as a change in global or regional climate patterns. Climate change has already been observed globally and in the United States. These included increases and changes in air and water temperatures, reduced frost days, increased frequency and intensity of heavy downpours, a rise in sea level, and reduced snow cover, glaciers, permafrost, and sea ice. Climate change has the potential to affect all of the missions of the United States Army Corps of Engineers (USACE). USACE mission in regards to climate change is: "To develop, implement, and assess adjustments or changes in operations and decision environments to enhance resilience or reduce vulnerability of USACE projects, systems, and programs to observed or expected changes in climate". The USACE's Climate Change Program develops and implements practical, nationally consistent, and cost-effective approaches and policies, to reduce potential vulnerabilities to the Nation's water infrastructure resulting from climate change and variability. The Department of the Army Engineering Regulation 1100-2-8162 (31 Dec 2013) requires that future Relative Sea Level Rise (RSLR) projections must be incorporated into the planning, engineering design, construction and operation of all civil works projects. The structural components of the proposed alternatives in consideration of the "low", "intermediate", and "high" potential rates of future RSLR were evaluated. This range of potential rates of RSLR is based on the findings of the National Research Council (NRC, 1987) and the Intergovernmental Panel for Climate Change (IPCC, 2007).

The same goes for hydrologic changes due to climate change. There is guidance for the H&H analysis to incorporate Climate Change. This USACE guidance ECB (Engineering and Construction Bulletin) 2018-14, dated 10 September 2018 and expires 10 September 2020, and is titled "Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects". This ECB "provides guidance for incorporating climate change information in hydrologic analyses in accordance with the USACE overarching climate change adaptation policy. This policy requires consideration of climate change in all current and future studies to reduce vulnerabilities and enhance the resilience of our water resources infrastructure." Hydrologic processes are very sensitive to changes in temperature, which can affect the form of precipitation (rain, snow), precipitation intensity and volume, the timing and volume of runoff, and conditions that cause or enhance drought. Observed climate change and variability have affected USACE water resources management-related missions and operations. USACE has developed and implemented policy and guidance to continue to provide reliable services in changing conditions. This chapter discusses the climate change impacts of and predictions for SLR and hydrology and how they influenced the design.

The Vaca Key gauge was used for the Sea Level Rise measurement. As mentioned before, the HH&C Sub Appendix Attachment 2 is a white paper prepared by the USACE Jacksonville District. The justification on why choosing the Vaca Key gauge for SLR is discussed in this attachment.

3.2 Components of Relative Sea Level Rise

RSLR considers the effects of (1) the eustatic, or global, average of the annual increase in water surface elevation due to the global warming trend, and (2) the "regional" rate of vertical land movement (VLM) that can result from localized geological processes, including the shifting of tectonic plates, the rebounding of the Earth's crust in locations previously covered by glaciers, the compaction of sedimentary strata and the withdrawal of subsurface fluids (USGS 2013).

The Miami-Dade County website states that Icecaps are melting in Greenland and as of the year 2017, global sea level rise is about 3.4 mm (0.13 inches) each year. There is also mention of another factor of sea level rise being the Changing Gulf Stream. The Changing Gulf Stream is when the current slows down or warms up and causing a local sea level rise effect on the east coast of Florida.

3.3Rates of Relative Sea Level Rise

When calculating the intermediate and high rates of RSLR, the local rate of VLM must first be determined. An example calculation of Miami-Dade County is provided in **Section 3.3.4** – Determining Local VLM. USACE published guidance on how to adapt to changing sea levels, Engineer Pamphlet 1100-2-1, Procedures to Evaluate Sea Level Change: Impacts, Responses and Adaptation. How RSLR is calculated is stated in the following paragraphs.

3.3.1 Historic Rate of Relative Sea Level Rise

The historic rate of future RSLR (or USACE Low Curve) is determined directly from gauge data gathered in the vicinity of the project area. The nearest NOAA tide gauge with over 30 years of data and from which tide data can be evaluated include: The Vaca Key, Florida Bay, FL gauge (NOAA Station 8723970). Tide conditions at Vaca Key indicate a relative sea level trend (RSLR + the local rate of VLM) of 3.66 +/- 0.44 mm/year (**Figure 3.1).**

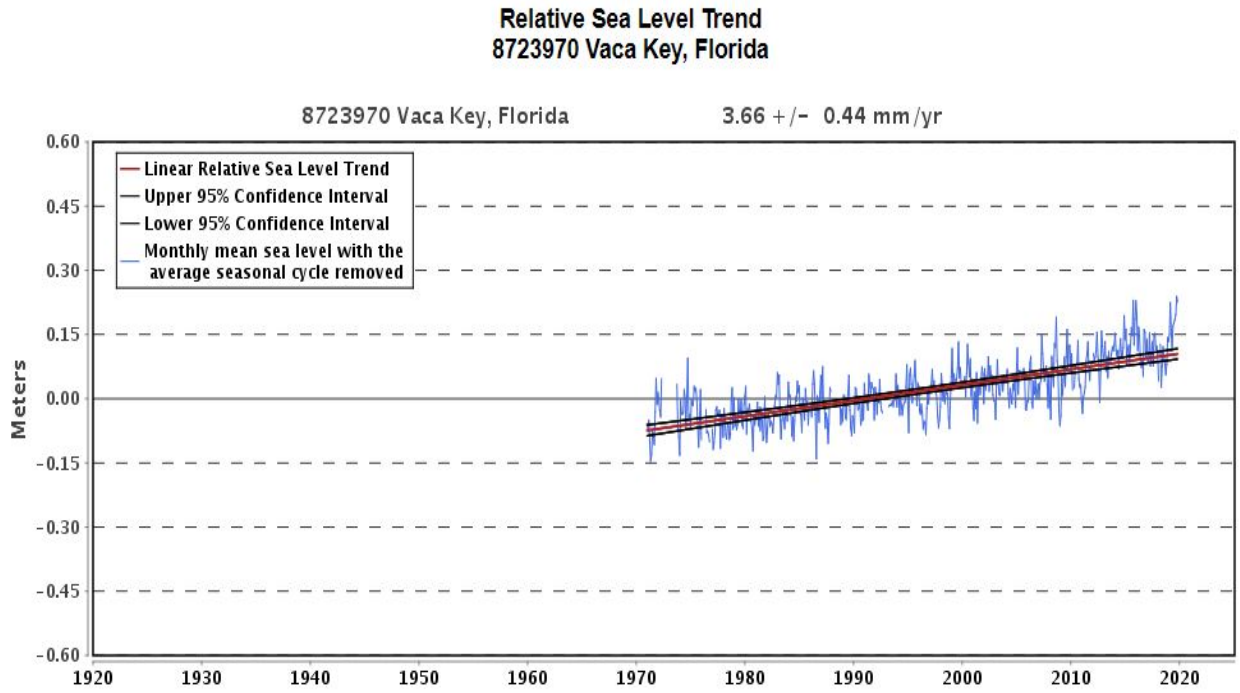


Figure 3.1. Relative Sea Level Trend at Vaca Key, Florida (NOAA Station # 8723970).

3.3.2 Intermediate Rate of Relative Sea Level Rise

The intermediate rate or "USACE Intermediate Curve" of local mean RSLR is estimated by considering the modified National Research Council (NRC) projections and adding the appropriate value to the local rate of vertical land movement. The intermediate rate of relative (local) sea level rise is based on the modified NRC Curve I (shown in green) (**Figure 3.2**) since its value is comparable to that of the IPCC projection.

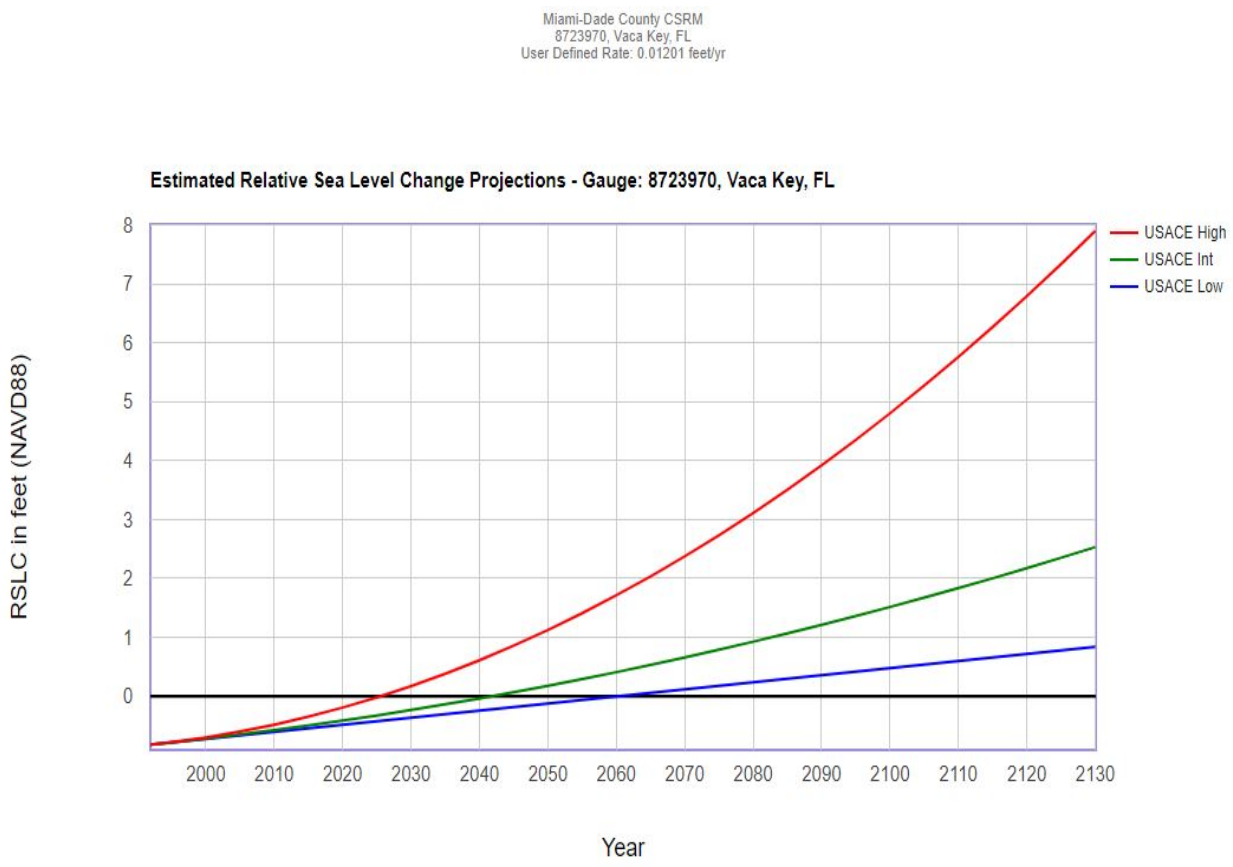


Figure 3.2 Modified NRC curves for predicting future rates of RSLR.

NRC Curve I is based on the general equation $E(t) = 0.0017t + bt^2$, where the constant 0.0017 = the IPCC 2007 annual rate of eustatic SLR in meters; t = time in years (relative to the year 1986 or the start date used by the NRC) and; $b = 2.71E^{-5}$.

3.3.3 High Rate of Relative Sea Level Rise

The USACE High Curve was used for the Miami-Dade County Back Bay CSRM Study. The high rate or "USACE High Curve" of mean RSLR is estimated by determining the modified NRC Curve III (shown in red) (**Figure 3.2**) value and adding it to the local rate of vertical land movement. This high rate scenario exceeds the 2001 and 2007 IPCC projections and considers the potential rapid loss of ice from Antarctica and Greenland. NRC Curve III is also based on the general equation $E(t) = 0.0017t + bt^2$; however, the constant b changes to $b = 1.13E^{-4}$.

For both the intermediate and high rates of RSLR, the NRC curves accelerate upward over time beginning in the year 1992 (which corresponds to the midpoint of the current National Tidal Datum Epoch of 1983-2001); therefore, it is necessary to estimate RSLR for a particular time horizon relative to 1992.

3.3.4 Determining Local VLM

The local rate of VLM, which is considered to be constant through time, is determined by subtracting the NRC/IPCC eustatic SLR value (1.7 mm/yr) from the local mean sea level trend. Recall that the two components figuring into the local mean sea level include the RSLR value and the local rate of VLM. The mean rate of RSLR at the Vaca Key station is +3.66 mm/year (approximately 7.20 inches in 50 years).

The local rate of VLM at Vaca Key is calculated from the relationship:

$$VLM_{Vaca\ Key} = [\text{local rate of RSLR}] - [\text{eustatic rate of SLR}], \text{ or}$$
$$VLM_{Vaca\ Key} = 3.66 \text{ mm/yr} - 1.7 \text{ mm/yr} = 1.96 \text{ mm/yr } (0.0772 \text{ in/yr})$$

At the Vaca Key gauge, the local rate of VLM accounts for a total of 3.86 inches (0.0772 in/yr x 50 yrs.) at a 50-year time horizon. This local rate of VLM is added back into the sea level rise computations after the eustatic portion has been determined from NRC curves I and III.

3.4 Calculating Relative Sea Level Rise

3.4.1 Historic Rate

The historic rate of relative sea level rise is determined by extrapolating the mean RSLR trend and multiplying it by the desired time horizon. The RSLR trend at Vaca Key is 3.66 mm/yr (0.144 inches in a year). Based on the historic rate of RSLR it can be expected that sea level will rise 7.2 inches over a 50-year time horizon.

3.4.2 Intermediate Rate

The intermediate rate of sea level rise is computed using the equation

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2) + \text{local VLM}$$

Where t_1 and t_2 represent the start and end dates of the projected time horizon in years, relative to 1992; therefore, t_1=38yr and t_2=88 yr (2030 and 2080 respectively) and b is a constant equal to 0.0000271 m/yr

$$RSLR = (0.0017\ m/yr\ (88-38) + 0.0000271\ m/yr\ (88^2-38^2))\ x\ (3.281\ ft/m) + (0.0772\ x\ 50\ yrs.)/12\ in/ft \approx 1.16\ ft.$$

This is stating from the start of the project economic period of analysis (2030) to the end of the project economic period of analysis (2079), the sea level is expected to rise approximately 1.16 feet, which relates to what has been calculated in **Table 3.1** and **Table 3.4.** The year 2080 was used in the equations for a 50 year project life.

3.4.3 High Rate

The high rate of RSLR is computed using the equation

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t2^2 - t1^2) + local\ VLM$$

Where t_1 and t_2 represent the start and end dates of the projected time horizon in years, relative to 1992; therefore, t_1=38yr and t_2=88 yr (2030 and 2080 respectively) and b is a constant equal to 0.000113 m/yr.

$$RSLR\ (High) = (0.0017\ (88-38) + 0.000113\ m/yr\ (88^2-38^2))\ x\ (3.281\ ft/m)) + (0.0772\ x\ 50\ yrs.)/12\ in/ft \approx 2.94\ ft.$$

This is stating from the start of the project economic period of analysis (2030) to the end of the project economic period of analysis (2079), the sea level is expected to rise approximately 2.94 feet, which relates to what has been calculated in **Table 3.1** and **Table 3.4**. The year 2080 was used in the equations for a 50 year project life.

3.5Projected Water Surface Level Increase

The Miami-Dade County Back Bay CSRM project design water level stages were derived from SFLSSS modeling efforts completed in 2017. **Table 3.1** shows the projected increase

in water surface elevation for the historic, intermediate, and high rates of future sea level rise at the Vaca Key, Florida Bay, FL gauge to the year 2130 (see **Figure 3.3** for illustration of future sea level rise and subsidence).

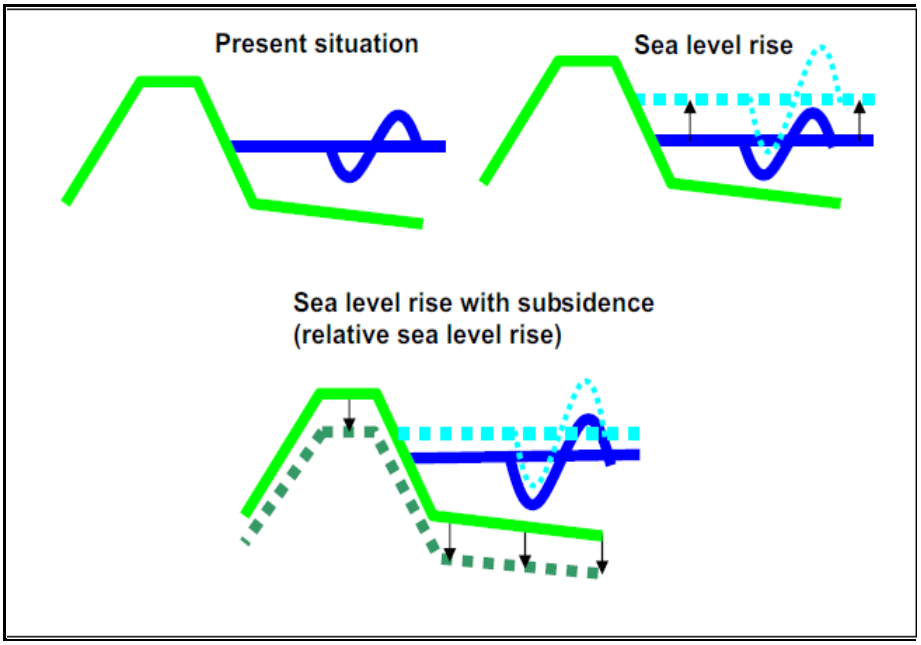


Figure 3.3 Illustrations and simple explanation of future sea level rise and subsidence.

Table 3.1 Increase in predicted water surface elevations at Vaca Key, Florida Bay, FL under the low, intermediate and high rates of future RSLR USACE Curves computed using criteria in USACE ER 1100-2-8162 (in NAVD88 feet).

Miami-Dade County CSRM
8723970, Vaca Key, FL
User Defined Rate: 0.01201 feet/yr
All values are expressed in feet relative to NAVD88

Year	USACE Low	USACE Int	USACE High
1992	-0.82	-0.82	-0.82
1995	-0.78	-0.78	-0.78
2000	-0.72	-0.72	-0.70
2005	-0.66	-0.65	-0.60
2010	-0.60	-0.57	-0.48
2015	-0.54	-0.50	-0.35
2020	-0.48	-0.41	-0.19
2025	-0.42	-0.33	-0.02
2030	-0.36	-0.24	0.17
2035	-0.30	-0.14	0.38
2040	-0.24	-0.04	0.61
2045	-0.18	0.07	0.86
2050	-0.12	0.18	1.12
2055	-0.06	0.29	1.41
2060	-0.00	0.41	1.71
2065	0.06	0.53	2.03
2070	0.12	0.66	2.37
2075	0.18	0.79	2.73
2080	0.24	0.93	3.11
2085	0.30	1.07	3.50
2090	0.36	1.21	3.92
2095	0.42	1.36	4.35
2100	0.48	1.51	4.80
2105	0.54	1.67	5.27
2110	0.60	1.84	5.76
2115	0.66	2.00	6.27
2120	0.72	2.17	6.79
2125	0.78	2.35	7.34
2130	0.84	2.53	7.90

The Sea Level Tracker tool was used to visualize the observed changes in sea level and to compare rends to the projected sea level changes per USACE Engineer Regulation 1100-2-8162 and Engineer Pamphlet (EP) 1100-2-1. The tool shows the historical, observed changes

in mean sea level (MSL) as measured sand reported for National Oceanic Atmospheric Administration (NOAA) tide gauges, mapped against the USACE sea level change (SLC) projections. Taken together, the tool enables the comparison of actual SLC with USACE SLC projections (as described in ER 1100-2-8162), along with observed monthly water levels and the computation of SLC trends based on historical data (Sant-Miller et al, 2018). **Figure 3.4** displays the results of this tool, comparing actual SLC for the 19-year (metonic cycle) midpoint moving average (dark blue line) and 5-year midpoint moving average (orange line) against the USACE SLC curve projections. The observed 19-year moving average is tracking along the intermediate SLC scenario while the 5-year moving average has been tracking nearer to the high scenario since 2013.

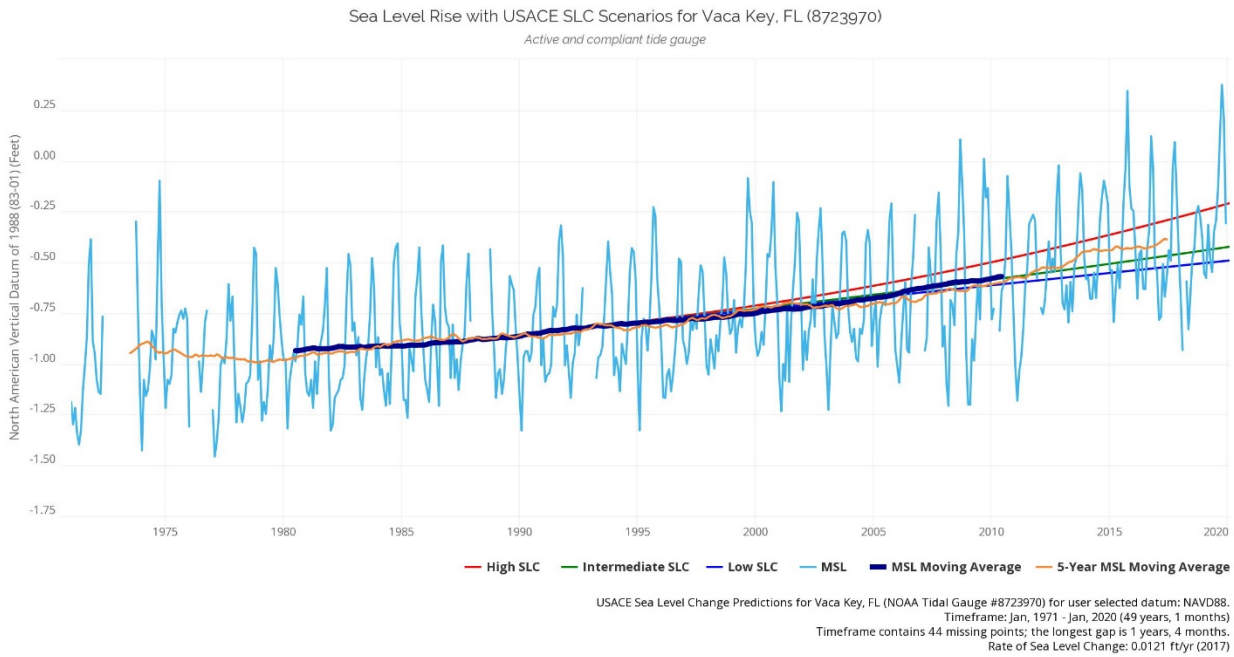


Figure 3.4. Historical Sea Level Rise with USACE SLC Scenarios for Vaca Key, FL (823970).

As stated earlier in this report, SWL includes tides, storm surge, and **Table 3.2** and **Table 3.3** show the SFLSSS SWL elevations for years 2018 (start of this study) and 2079 (which includes RSLR), respectively. The tables also show the four frequencies (5% AEP (20 year), 2% AEP (50 year), 1% AEP (100 year), 0.5% AEP (200 year)) which were chosen to be

analyzed. This assumes relative sea level rise to 2079 based on the USACE high curve. The PDT decided that the SFLSSS 1% AEP (100 year) SWL elevation should be in the range of elevations used for formulating separable measures throughout Miami-Dade County. The SWL elevation associated with the 5% AEP (20-yr) is the lower limit of the analysis. The 2% AEP (50 year), 1% AEP (100 year), and 0.5% AEP (200 year) events were also analyzed.

Table 3.2. SFLSS peak SWL elev. (90CL) (in feet, NAVD88) w/o RSLR in 2018.

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)**	**Arch Creek (Save Point 615)**
20	5	4.37	3.86	3.8	3.89	3.78
50	2	6.09	5.14	5.04	5.24	5.07
100	1	7.31	6.15	5.95	6.19	6.13
200	0.5	8.66	7.44	6.88	7.12	7.71

Table 3.3. SFLSS peak SWL elev. (90CL) (in feet, NAVD88) w/ RSLR to 2079 (High 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)**	**Arch Creek (Save Point 615)**
20	5	7.66	7.15	7.09	7.18	7.07
50	2	9.38	8.43	8.33	8.53	8.36
100	1	10.6	9.44	9.24	9.48	9.42
200	0.5	11.95	10.73	10.17	10.41	11.00

To get a perspective of the amount of Sea Level Rise from the start of this study through the 50 year project life, **Table 3.4a** shows the total SLR increase based on the appropriate USACE SLR curve using the Vaca Key, Biscayne Bay, FL NOAA tide gauge.

Table 3.4a. SLR adjustment values up to 2079.

VACA KEY	USACE Low Curve		USACE Intermediate Curve		USACE High Curve	
SLR Adjustments	[ft]	[m]	[ft]	[m]	[ft]	[m]
1992 to 2018	0.310	0.095	0.370	0.113	0.560	0.171
2018 to 2030	0.150	0.046	0.220	0.067	0.430	0.131
2030 to 2079	0.590	0.180	1.130	0.345	2.860	0.872
2018 to 2079	0.740	0.226	1.350	0.412	3.290	1.003

3.6Impacts of Sea Level Rise during Future Years

Engineering Pamphlet (EP) 1100-2-1, Global Changes – Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation, recommends analyzing the effects of SLR on the project at three future time periods post construction (year 2030). The time periods include 20 years after construction is completed (the calendar year 2050), 50 years after construction (the economic year 2079), and 100 years after construction (the calendar year 2130). This section will discuss RSLR on the project at the year 2079, which is the end of the project economic period of analysis and briefly discuss the RSLR rates over the project economic period of analysis for the 5% AEP (20 year), 2% AEP (50 year), 1% AEP (100 year) and 0.5% AEP (200 year) and its potential impacts. **Table 3.4** shows the predicted increase in SLR for the computed USACE curves 20 years (2050), 50 years (2079), and 100 years (2130) into the future after the proposed construction year of 2030.

Table 3.2. Increase in SLR predicted for 20 years, 50 years, and 100 years into the future (in feet) from the project start year of 2030 at the NOAA Vaca Key, Florida Bay, FL tide gauge.

Year	USACE Low	USACE Int.	USACE High	Reference Section
2030 to 2050	0.24	0.42	0.95	Table 3.1
2030 to 2079	0.59	1.13	2.86	Table 3.1
2030 to 2130	1.2	2.77	7.73	Table 3.1

The values in **Table 3.4** for the year 2050, 2079 and 2130, were derived from **Table 3.1** in this sub-appendix.

Table 3.5 to Table 3.7, display the SWLs with the predicted increase in SLR from 2018 to 2130 using the Low, Intermediate and High USACE SLR curves. **Table 3.8** has the SWL with the predicted increase in SLR from 2018 to 2130 using the NOAA high SLR curve. In the HH&C Sub Appendix Attachment 2 are additional SLR tables for the years 2030, 2050, 2079, and 2130 using three USACE SLR curves and one NOAA Curve.

Table 3.3. Peak SWL elev. (90 CL) (in feet, NAVD88) w/RSLR at 2130 (USACE Low 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)**	**Arch Creek (Save Point 615)**
20	5	5.72	5.21	5.15	5.24	5.13
50	2	7.44	6.49	6.39	6.59	6.42
100	1	8.66	7.50	7.30	7.54	7.48
200	0.5	10.01	8.79	8.23	8.47	9.06

Table 3.6. Peak SWL elev. (90 CL) (in feet, NAVD88) w/RSLR at 2130 (USACE Int 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)**	**Arch Creek (Save Point 615)**
20	5	7.35	6.84	6.78	6.87	6.76
50	2	9.07	8.12	8.02	8.22	8.05
100	1	10.29	9.13	8.93	9.17	9.11
200	0.5	11.64	10.42	9.86	10.10	10.69

Table 3.7. Peak SWL elev. (90 CL) (in feet, NAVD88) w/RSLR at 2130 (USACE High 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)**	**Arch Creek (Save Point 615)**
20	5	12.53	12.02	11.96	12.05	11.94
50	2	14.25	13.30	13.20	13.40	13.23
100	1	15.47	14.31	14.11	14.35	14.29
200	0.5	16.82	15.60	15.04	15.28	15.87

Table 3.8. Peak SWL elev. (in feet, NAVD88) w/ RSLR at 2130 (NOAA High 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)	Arch Creek (Save Point 615)
20	5	15.10	14.59	14.53	14.62	14.51
50	2	16.82	15.87	15.77	15.97	15.80
100	1	18.04	16.88	16.68	16.92	16.86
200	0.5	19.39	18.17	17.61	17.85	18.44

3.7Climate Change Impacts to Hydrology

The USACE January 2015 Civil Works Technical Report, CWTS-2015-03 for the South Atlantic-Gulf Region of the U.S., focuses on temperature, extreme precipitation events, and stream flow trends and future findings. The report encompasses the HUC 03 region and also indicates the HUC 03 boundaries within (see **Figure 3.6**). The report states that trends in temperature show a mild upward trend, a mild downward trend in streamflow since the 1970s, and no observed changes in hydrology. The report states that precipitation show mixed results and more findings showing an upward pattern over the past 50 to 100 years. This is summarized in **Figure 3.7** which is a matrix of observed and projected climate trends from CWTS-2015-03.

If precipitation increases as projected in **Figure 3.6**, potential project vulnerabilities may include increased interior rainfall runoff leading to increased rainfall flooding. This may require increased pump capacity and longer pumping times, causing increased maintenance cost. Interior flooding may also increase operational complexity if operators have more difficulty navigating flooded streets to access project sites. For the future trends, the report states that there is a strong consensus the air temperature will increase, a moderate consensus that future storm events will be more frequent and intense, and the hydrology showed peak flow increases and peak flow decreases.

Locally, the official climatological gage for Miami is located at the Miami International Airport. The station name is Miami International Airport, FL US (USW00012839). **Figure**

3.8 and **Figure 3.9** represent the linear trends in annual total precipitation and annual average temperatures, respectively, over the period 1948-2019.

Figure 3.5. TR reference map.

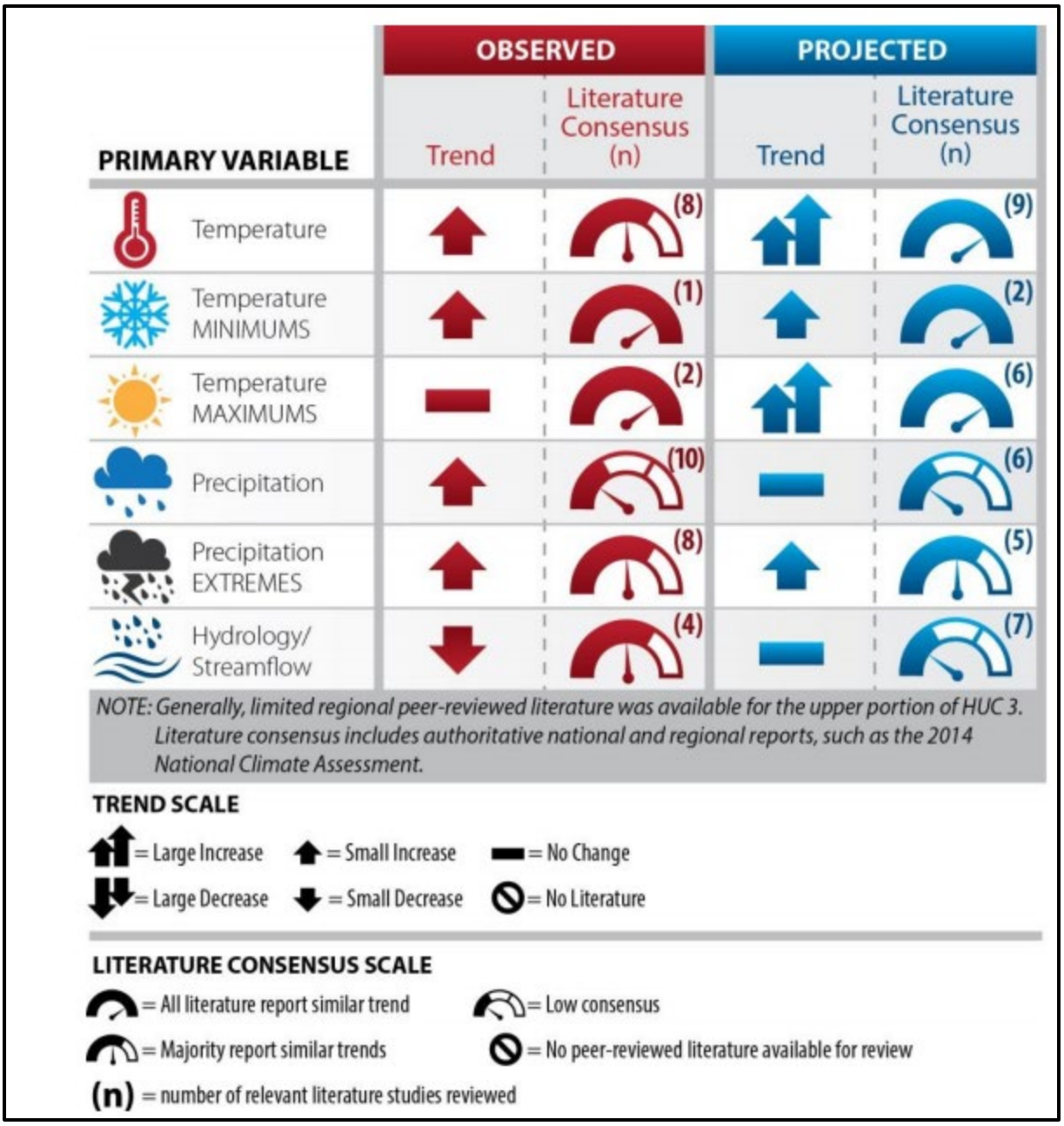


Figure 3.6. Summary matrix of observed and projected climate trends from CWTS-2015-03.

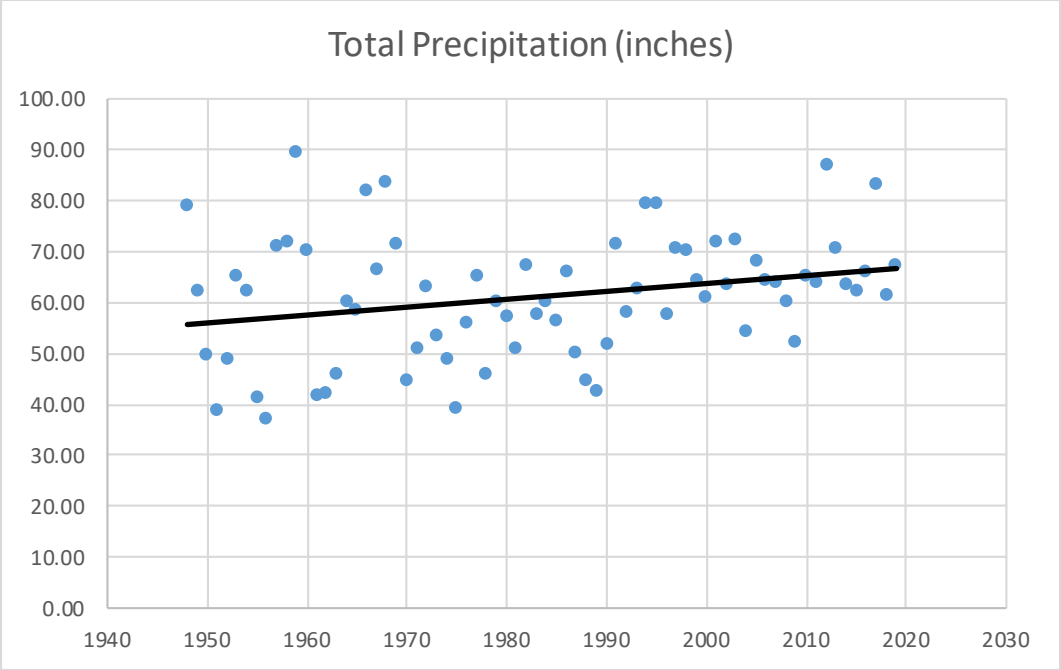


Figure 3.7. Precipitation Trend for Miami-Dade County.

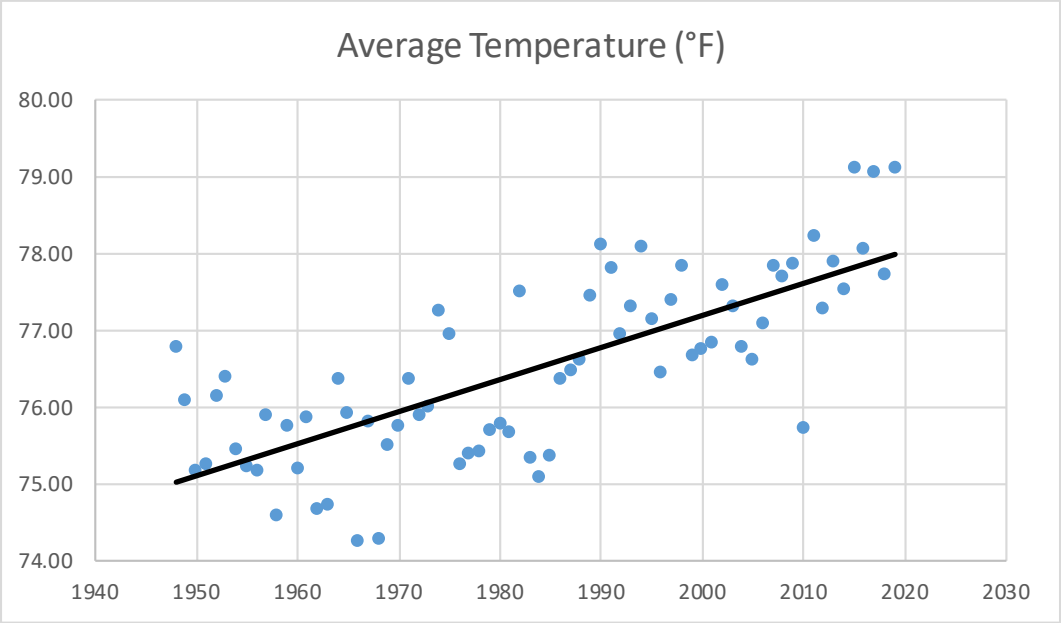


Figure 3.8. Temperature Trend for Miami-Dade County.

The ECB (Engineering and Construction Bulletin) 2018-14 includes the reference to the Climate Hydrology Assessment tool. From the ECB, "The Climate Hydrology Assessment

tool allows users to access data concerning past (observed) changes as well as potential future (projected) changes to relevant hydrologic inputs. This provides qualitative information about future climate conditions useful to decision-making officials, and allows districts across the country to develop repeatable analytical results using consistent information. The tool reduces potential error while increasing the speed of information development so that data can be used earlier in the decision-making process, ideally in the development of risk registers."

For the Hydrology Tools, both the Nonstationarity Detection Tool and the Climate Hydrology Assessment Tool have USGS stream gauges that are listed per HUC watershed. For this study area this tool is not applicable because there were no USGS stream gauges listed in the database for the study area.

At this point in the study, Hydrologic Climate change for this study area was addressed by using one frequency (the 100 year) for the Peak Flood Analyses. The rainfall events for the 2-, 5-, 10-, 25-, 50-, 100-, and 500- year frequencies may be included before the Final Report of this HH&C Sub Appendix. With this method, an array of flows and storage levels in the USACE HEC-HMS Hydrologic Modeling System program can show the potential of such large rainfall events. Please refer to **Chapter 7** of this report for information on the H&H analysis performed for this study.

The rainfall amounts used for input into the hydrologic model are from the NOAA Atlas 14 Vol. 9 Version 2.0, which are the point precipitation frequency estimates. From the NOAA Atlas 14 report, "The 1-day annual maximum series were analyzed for linear trends in mean and variance and shifts in mean to determine whether climate change during the period of record was an issue in the production of this atlas (Appendix A.3). The results showed little observable or geographically consistent impact of climate change on the annual maximum series during the period of record and so the entire period of record was used. The estimates presented in this atlas make the necessary assumption that there is no effect of climate change in future years on precipitation frequency estimates. The estimates will need to be modified if that assumption proves quantifiably incorrect."

3.8 Adaptation of the Components of the Recommended Plan for Climate Change

The Miami-Dade County Back Bay CSRM Project consists of components that are adaptable to future increases in sea level due to climate change. Currently, 3.29 feet (using the USACE high SLR curve) were added to the SWL elevations to account for the SLR anticipated for the year 2018 to the year 2079. If the predicted rate of SLR changes (increases), the wall systems could also be modified with parapet walls or additional wave baffles pending design analyses to support additional height. If applicable, additional pump station capacity could be added to handle additional overtopping and interior drainage from precipitation. In Miami-Dade County the level of groundwater is another tidally influenced component and with sea level rise, groundwater could potentially affect the structural measures proposed.

CHAPTER 4 Wave Data, Modeling, and Results

This chapter will discuss the frequency wave data computed by ERDC used from the SFLSSS and how the data influenced the significant wave heights chosen to perform the wave overtopping analysis and wave forces on a vertical wall for the different Stillwater elevations.

4.1SFLSSS Wave Data

The SFLSSS modeling effort, produced not only computed SWL elevations for different frequencies (see **Chapter 2** and **Chapter 3** of this sub-appendix), but the significant wave heights (AEP significant wave heights) for each storm frequency as well. The SFLSSS numerical modeling study produced wave and water level estimates and the associated marginal and joint probabilities. Data (storms, waves, tides, etc.) was collected from many resources. Once storms were selected and the necessary data input into the ADCIRC model was used to simulate the surge and circulation response to the storms; and SWAN was used to provide the nearshore wave conditions including local wind generated waves. ADCIRC is a system of computer programs for solving time dependent, free surface circulation and transport problems in two and three dimensions. This model utilizes the finite element method in space allowing the use of highly flexible, unstructured grids. Typical ADCIRC applications have included:

- prediction of storm surge and flooding
- modeling tides and wind driven circulation
- larval transport studies
- near shore marine operations
- dredging feasibility and material disposal studies

The SWAN model runs on the same triangulated mesh that is used with the ADCIRC model. During the model simulations, the water levels from ADCIRC are fed into the SWAN model at 15-minute interval (of model time). The SWAN model computes the wind-driven development of the storm waves, the propagation of the waves over the model domain, and the wave radiation stress gradients where the waves break close to the shore.

Table 4.1 shows the significant wave heights and **Table 4.2** shows the frequency wave periods for the save points used near the surge barriers for the five areas of structural measures (Arch Creek is pending). The tables show four frequencies at the 90% Confidence Limit (90CL) (5% AEP (20 year), 2% AEP (50 year), 1% AEP (100 year), and 0.5% AEP (200 year) which were chosen to be analyzed.

Table 4.1 The significant wave heights for the save points used near the surge barriers.

SFLSSS Significant Wave Height (in feet) - 90CL						
Annual Recurrence Interval (years)	Annual Exceedance Probability	**Miami River (Save Point 443)**	**Edgewater (Save Point 572)**	**Little River (Save Point 614)**	**Biscayne Canal (Save Point 621)**	**Arch Creek (Save Point 615)**
20	5%	5.95	3.32	2.7	1.94	2.77
50	2%	6.73	4.04	3.5	2.77	4.00
100	1%	7.25	4.51	4.2	3.11	4.68
200	0.50%	7.73	4.96	5	4.17	5.23

Table 4.2 The frequency wave periods for the save points used near the surge barriers.

SFLSSS Frequency Wave Period (in seconds)- 90CL						
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)**	**Arch Creek (Save Point 615)**
20	5%	3.94	3.48	3.46	3.13	3.18
50	2%	4.4	3.76	3.91	3.59	5.56
100	1%	4.61	3.93	4.16	3.92	6.96
200	0.50%	4.77	4.08	4.39	4.17	8.44

The HH&C analysis used the corresponding frequency wave period and significant wave height for the SWL elevation frequencies analyzed. Most of the selected save points have a correlation of at least 0.5, signifying that the assumption of using the 1% AEP SWL elevation correlates to the same Hs AEP, i.e. the 100-yr correlates to the 100-yr Hs, is a very conservative approach. The significance of a save point with a correlation in the 0.70s or 0.80s is that the magnitude of the wave height could be on the conservative side relative to

the corresponding SWL AEP. If correlation << 1, it is expected that the magnitudes of the wave height AEP would be less than the corresponding SWL AEP, i.e. for a 50-yr SWL the expected wave would be less than 50-yr (example: Hs = 30 or 40yr, etc.). See **Table 4.3** for the wave height correlation ratios.

Table 4.3 Wave Height Correlations

SvPt ID	Latitude (deg)	Longitude (deg)	ADCIRC node	SWL-Hs Correlation
443	25.7701	-80.1879	1303455	0.45
572	25.8036	-80.1862	1526185	0.54
614	25.8449	-80.1734	1629089	NaN
615	25.8987	-80.1612	1629322	0.03
621	25.8718	-80.1647	1637144	0.44

CHAPTER 5 Wave Overtopping Analysis

There are two types of overtopping: Flood Overtopping and Wave Overtopping. Flood Overtopping is when a continuous flow of a pool elevation or river elevation exceeds the low portions of a levee and floodwall. For overtopping by waves (or Wave Overtopping), the Stillwater elevation approaches but does not exceed the low point in the elevation profile. Instead, waves driven by wind produce waves that run-up and overtop the top of a levee or floodwall. The wave action can form an equivalent discharge per a linear distance of the structure and can lead to the erosion, potential failure of the structure and can create ponding areas on the land-side of a project alignment if pump stations are not considered. Waves are influenced by wind speed, wind direction, bathymetry, open water distance, and embankment slopes. Wave overtopping of berms and vertical walls are of concerned for coastal areas, which is why it is being calculated for Miami-Dade County.

Wave overtopping was analyzed using the frequency SWL elevation and the frequency wave heights from the SFLSSS wave data. The analysis included the development of overtopping rates for the return periods 5% flood, 2% flood, 1% flood, and 0.5% flood (20-yr, 50-yr, 100-yr, and 200-yr) at SWL elevations calculated for the year 2079 water levels (end of the project economic period of analysis) and for year 2130 (50 years from the end of the project economic period of analysis), which vary slightly per study area within Miami-Dade County. The comparison of different methods for overtopping computation show that the choice of method is not highly influential on the design. The wave heights from existing data (mentioned in **Chapter 4** of this sub-appendix) varied per storm and duration of the storm, however, the average height per SWL elevation was used as the deep-water significant wave height (H_o) for the overtopping analysis for each storm frequency. The height and elevation of the structural measures were determined by first assuming that the top elevation of the structural measures equal to the SWL elevations then analyzing the wave overtopping. The final elevations shown in the summary tables in this chapter were the height of the structural measures that allowed for minimum wave overtopping throughout the economic period of analysis of the project.

The risk-reducing capability of the Miami-Dade County Back Bay CSRM project during coastal storms is dependent upon the barriers and floodwalls ability to resist against wave overtopping flow rates. Waves breaking may result in water splashing over the berm/levee crest or top of the structural measures onto the landward side of the protection when the Stillwater surface elevation is lower than the crest elevation of the barrier or floodwall. These wave overtopping rates have the potential of causing scour and possibly failure of the landside of the structural measures. The overtopping analysis, for the designed vertical wall and barriers, 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 EurOtop (which is based off equations that can be found in the EurOtop Overtopping Manual) were utilized for comparison. The EurOtop (which stands for the European Overtopping) Overtopping Manual was developed by a company named HR Wallingford out of the Netherlands and Germany.

EurOtop Overtopping Manual incorporates new techniques to predict wave overtopping at seawalls, flood embankments, breakwaters and other shoreline structures facing waves and uses both the Probabilistic and Deterministic approaches to analyze the overtopping for the design of the vertical wall and levee.

5.1Deterministic Versus Probabilistic Approach

As mentioned in the previous section, the EurOtop method used both a deterministic and probabilistic approaches to analyze the wave overtopping rates for the design of the vertical wall. Design conditions for major coastal and flood protection projects are often vague and design parameters contain large uncertainties. Imposed forces, as well as the strengths and interactions of the various components are usually not clearly understood and the design process itself is ill defined. In the past, designs were strictly based on deterministic expressions. A deterministic model is one in which every set of variable states is uniquely determined by parameters in the model and by sets of previous states of these variables. Therefore, deterministic models perform the same way for a given set of initial conditions. More recently, probabilistic design methods have been introduced, in which randomness is

present and variable states are not described by unique values, but rather by probability distributions. Both approaches are typically compared.

5.2Overtopping Floodwall Analysis

The overtopping rates were calculated for the project flood wall using the overtopping formulations provided in ACES software and the EurOtop methods.

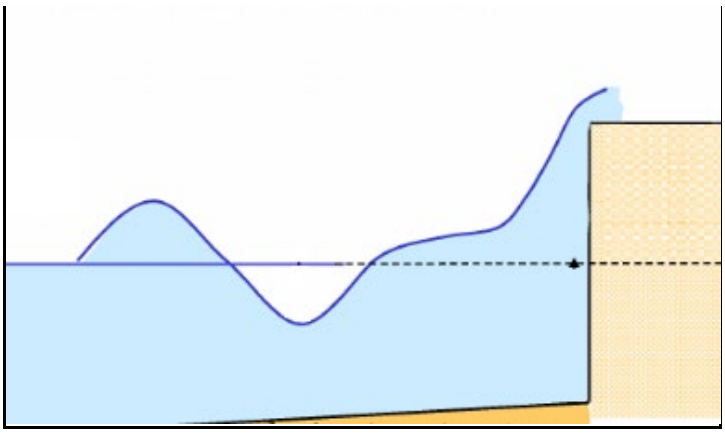


Figure 5.1. Image Representing Wave Overtopping of a Vertical Wall.

The equations, formulations used, and results from each method are shown and explained throughout this section.

5.2.1 ACES Software

ACES is an interactive computer-based design and analysis system in the field of coastal engineering containing six functional areas: wave prediction, wave theory, wave transformation, structural design, wave run-up, and littoral processes. For the purpose of this analysis ACES was used to calculate wave-overtopping for the project flood wall conditions. Below (**Figure 5.2**) is an image of the ACES interactive interface for solving for wave-overtopping:

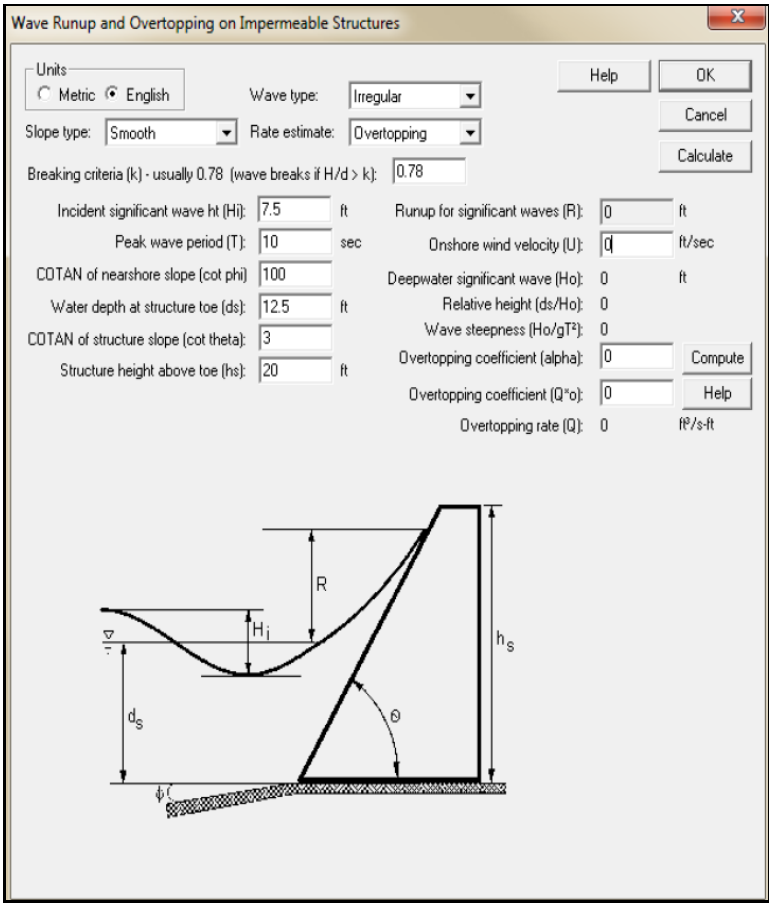


Figure 5.2 ACES interface. Note: Values shown in the interface were not used in this analysis.

The incident significant wave height (H_i), peak wave period (T), COTAN of nearshore slope (cot phi), water depth at the structure toe (d_s), COTAN of structure slope (cot theta), structure height above toe (h_s), and onshore wind velocity (U) (which we will assume to be 80 ft/sec for all calculations) are input into the ACES interface. The Overtopping coefficient (alpha) is computed in ACES based off of the COTAN of structure slope value, which is zero for the vertical wall. The Run-up for significant waves (R), Deepwater significant wave (Ho), Relative height (d_s/H_o), and the Wave steepness (Ho/gT2) are all calculated from the equations programmed in ACES. The Overtopping Coefficient (Q^*_o) can be found by using Figures 7-24, in the Shore Protection Manual (SPM). Results are shown in **Section 5.2.3** Individual Model Results of Floodwall Overtopping Analysis.

5.2.2 EurOtop

The overtopping rates were also analyzed using EurOtop. Below (Figure 5.3) is an image of the deterministic and probabilistic equations used in calculating the wave overtopping.

Deterministic Equations

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b\xi_{m-1,0}\exp\left(-4.3\frac{R_c}{\xi_{m-1,0}H_{m0}\gamma_b\gamma_f\gamma_\beta\gamma_v}\right)$$

$$\frac{q}{\sqrt{gH_{m0}^3}} < 0.2\cdot\exp\left(-2.3\frac{R_c}{H_{m0}\gamma_f\gamma_\beta}\right)$$

Wave overtopping for short-crested waves

$$\gamma_\beta = 1 - 0.0033|\beta| \qquad 0° \le |\beta| \le 80°$$

$$\gamma_\beta = 1 - 0.0033(80) \qquad |\beta| > 80°$$

Probabilistic Equations

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b\xi_{m-1,0}\exp\left(-4.75\frac{R_c}{\xi_{m-1,0}H_{m0}\gamma_b\gamma_f\gamma_\beta\gamma_v}\right)$$

$$\frac{q}{\sqrt{gH_{m0}^3}} < 0.2\cdot\exp\left(-2.6\frac{R_c}{H_{m0}\gamma_f\gamma_\beta}\right)$$

Uncertainty
$\mu = 4.75, \sigma = 0.5$
$\mu = 2.6, \sigma = 0.35$

Figure 5.3 Equations used in the EurOtop method.

The Wave Height, Wave Period, Total Depth at Structure, bottom slope, the Roughness Influence Factor, Acceleration of Gravity, Wave Obliquity, and the Breaker Ratio for breaking waves are all factors that are inputted in the EurOtop calculation of the wave overtopping. Overtopping Rates and Run-up for significant waves were computed.

Results are shown in **Section 5.2.3** Individual Model Results of Floodwall Overtopping Analysis.

5.2.3 Individual Model Results of Floodwall Overtopping Analysis

Table 5.1 to Table 5.12 compare the results (for adaptability) of each method used to calculate overtopping rates for the structural measures at the AEP 5%, 2%, 1%, and 0.5% (20 year, 50 year, 100 year, and 200 year) SWL elevations calculated for year 2079 (end of the project economic period of analysis) and for year 2079 using the NOAA high SLR curve. In the comparison tables, there are a few cases where the floodwall designed for the year 2079 SWL elevation plus SLR will show submerged, where the depth of water at the toe of the floodwall was greater than the height of the wall, using the NOAA high SLR curve. The next section**, Section 5.3**, discusses the design allowable rate of overtopping. The selected design rate of allowable overtopping was for the "no damage" condition for the heights analyzed. Under the "no damage" condition, most of the results shown for the NOAA high SLR curve indicate the allowable overtopping rate is exceeded. The overtopping analysis was also calculated using the 2130 SWLs for the year 2079 top of wall elevations and indicated submerged structures for most of the alignments as expected. The red lined rows in **Tables 5.1 to 5.12** are indicating no wall needed as the ground elevation was higher than the frequency SWL. Based on the adaptability comparisons, the proposed top of wall SWL elevations for the year 2079 will require additional wall height to meet the "no damage" condition using the NOAA high SLR curve SWL elevations and the 2130 USACE high curve SWL elevations.

For the surge barrier structures, the wave data presented in **Table 4.1** was used as the inputs into ACES and the EurOtop spreadsheets. For the floodwalls on the land, no wave data save point was close enough to associate with a portion of the wall (i.e. south wall, north wall, etc.). A rule of thumb was used where the significant wave height (Hi) was equal to two thirds of the water depth at the toe (Ds) to define a correlating wave height for the floodwalls on land. Attachment HH&C-1 at the end of this sub-appendix shows

output tables (from ACES) and EurOtop spreadsheets, which also display all input parameters and calculated solutions for both methods.

Table 5.1a. Overtopping Analysis of South Floodwall for Miami River (in feet).

		Overtopping Analysis of Flood Wall				
		Miami River Option 1 South Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	7.66	0.50	0.0120	0.0031	0.0020
2	50	9.38	3.50	0.0190	0.1218	0.0862
1	100	10.6	6.00	0.0150	0.1716	0.1157
0.5	200	11.95	9.00	0.0130	0.2042	0.1315
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	8.62	0.50	submerged	submerged	submerged
2	50	10.34	3.50	0.5900	1.6142	1.4816
1	100	11.56	6.00	0.2790	1.1354	0.9303
0.5	200	12.91	9.00	0.1560	0.8955	0.6732

Table 5.1b. Overtopping Analysis of South Floodwall for Miami River (in meters).

		Overtopping Analysis of Flood Wall				
		Miami River Option 1 South Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	NACCS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	7.66	0.50	1.11	0.29	0.19
2	50	9.38	3.50	1.77	11.32	8.01
1	100	10.60	6.00	1.39	15.94	10.75
0.5	200	11.95	9.00	1.21	18.97	12.22
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	NACCS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	8.62	0.50	submerged	submerged	submerged
2	50	10.34	3.50	54.81	149.96	137.65
1	100	11.56	6.00	25.92	105.48	86.43
0.5	200	12.91	9.00	14.49	83.19	62.54

Table 5.2a. Overtopping Analysis of South Floodwall in Water for Miami River (in feet).

		Overtopping Analysis of Flood Wall				
		Miami River Option 1 South Wall in Water				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	7.66	19	0.017	0.4702	0.3165
2	50	9.38	22	0.020	0.6033	0.4040
1	100	10.6	24	0.020	0.6632	0.4414
0.5	200	11.95	26.5	0.015	0.6312	0.4104
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	8.62	19	0.050	0.7954	0.5735
2	50	10.34	22	0.050	0.9302	0.6590
1	100	11.56	24	0.051	0.9837	0.6892
0.5	200	12.91	26.5	0.035	0.9064	0.6179

Table 5.2b. Overtopping Analysis of South Floodwall in Water for Miami River (in meters).

		Overtopping Analysis of Flood Wall				
		Miami River Option 1 South Wall in Water				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	7.66	19.00	1.58	43.68	29.40
2	50	9.38	22.00	1.86	56.05	37.53
1	100	10.60	24.00	1.86	61.61	41.01
0.5	200	11.95	26.50	1.39	58.64	38.13
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	8.62	19.00	4.64	73.90	53.28
2	50	10.34	22.00	4.65	86.42	61.22
1	100	11.56	24.00	4.74	91.39	64.03
0.5	200	12.91	26.50	3.28	84.21	57.40

Table 5.3a. Overtopping Analysis of Surge Barrier for Miami River (in feet).

		Overtopping Analysis of Flood Wall				
		Miami River Option 1 Surge Barrier				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	7.66	15.9	0.020	0.6814	0.4498
2	50	9.38	18.9	0.018	0.7656	0.5008
1	100	10.6	20.9	0.019	0.8441	0.5512
0.5	200	11.95	23.4	0.015	0.8084	0.5184
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	8.62	15.9	0.046	0.9876	0.6842
2	50	10.34	18.9	0.040	1.0629	0.7257
1	100	11.56	20.9	0.041	1.1446	0.7777
0.5	200	12.91	23.4	0.031	1.0757	0.7160

Table 5.3b. Overtopping Analysis of Surge Barrier for Miami River (in meters).

		Overtopping Analysis of Flood Wall				
		Miami River Option 1 Surge Barrier				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	7.66	15.90	1.86	63.30	41.79
2	50	9.38	18.90	1.67	71.13	46.53
1	100	10.60	20.90	1.77	78.42	51.21
0.5	200	11.95	23.40	1.39	75.10	48.16
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	8.62	15.90	4.25	91.75	63.56
2	50	10.34	18.90	3.72	98.75	67.42
1	100	11.56	20.90	3.84	106.34	72.25
0.5	200	12.91	23.40	2.88	99.94	66.52

Table 5.4a. Overtopping Analysis of North Floodwall for Miami River (in feet).

		Overtopping Analysis of Flood Wall				
		Miami River Option 1 North Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	7.66	9.5	0.010	0.1038	0.0707
2	50	9.38	13	0.016	0.1900	0.1249
1	100	10.6	15.5	0.016	0.2586	0.1678
0.5	200	11.95	18.5	0.015	0.3045	0.1928
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	8.62	9.5	0.422	1.2866	1.1333
2	50	10.34	13	0.215	0.9912	0.7742
1	100	11.56	15.5	0.159	0.9781	0.7300
0.5	200	12.91	18.5	0.105	0.9231	0.6574

Table 5.4b. Overtopping Analysis of North Floodwall for Miami River (in meters).

		Overtopping Analysis of Flood Wall				
		Miami River Option 1 North Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	7.66	9.50	0.93	9.64	6.57
2	50	9.38	13.00	1.49	17.65	11.60
1	100	10.60	15.50	1.49	24.02	15.59
0.5	200	11.95	18.50	1.39	28.29	17.91
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	8.62	9.50	39.21	119.53	105.29
2	50	10.34	13.00	19.97	92.09	71.93
1	100	11.56	15.50	14.77	90.87	67.82
0.5	200	12.91	18.50	9.75	85.76	61.07

Table 5.5a. Overtopping Analysis of Edgewater Floodwall (in feet).

		Overtopping Analysis of Flood Wall				
		Edgewater Option Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	7.15	9.1	0.009	0.0689	0.0450
2	50	8.43	11.6	0.011	0.1407	0.0916
1	100	9.44	13.6	0.015	0.2035	0.1321
0.5	200	10.73	16.1	0.019	0.3059	0.1991
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	8.11	9.1	0.318	1.0418	0.8987
2	50	9.39	11.6	0.204	0.9282	0.7362
1	100	10.4	13.6	0.164	0.9357	0.7129
0.5	200	11.69	16.1	0.146	1.0357	0.7662

Table 5.5b. Overtopping Analysis of Edgewater Floodwall (in meters).

		Overtopping Analysis of Flood Wall				
		Edgewater Option Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	7.15	9.10	0.84	6.40	4.18
2	50	8.43	11.60	1.02	13.07	8.51
1	100	9.44	13.60	1.39	18.91	12.27
0.5	200	10.73	16.10	1.77	28.42	18.50
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	8.11	9.10	29.54	96.79	83.49
2	50	9.39	11.60	18.95	86.23	68.40
1	100	10.4	13.60	15.24	86.93	66.23
0.5	200	11.69	16.10	13.56	96.22	71.18

Table 5.6a. Overtopping Analysis of Edgewater Floodwall in Water (in feet).

		Overtopping Analysis of Flood Wall				
		Edgewater Option Wall in Water				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	7.15	11.4	0.015	0.3614	0.2461
2	50	8.43	13.9	0.012	0.4093	0.2727
1	100	9.44	15.4	0.017	0.5202	0.3500
0.5	200	10.73	17.4	0.018	0.5688	0.3799
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	8.11	11.4	0.059	0.7028	0.5220
2	50	9.39	13.9	0.043	0.7070	0.5058
1	100	10.4	15.4	0.052	0.8488	0.6087
0.5	200	11.69	17.4	0.049	0.8877	0.6285

Table 5.6b. Overtopping Analysis of Edgewater Floodwall in Water (in meters).

		Overtopping Analysis of Flood Wall				
		Edgewater Option Wall in Water				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	7.15	11.40	1.39	33.58	22.86
2	50	8.43	13.90	1.11	38.03	25.33
1	100	9.44	15.40	1.58	48.33	32.52
0.5	200	10.73	17.40	1.67	52.84	35.29
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	8.11	11.40	5.48	65.29	48.50
2	50	9.39	13.90	3.99	65.68	46.99
1	100	10.4	15.40	4.83	78.86	56.55
0.5	200	11.69	17.40	4.55	82.47	58.39

Table 5.7a. Overtopping Analysis of Little River South Floodwall (in feet).

		Overtopping Analysis of Flood Wall				
		Little River Option South Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	7.09				
2	50	8.33				
1	100	9.24	10	0.017	0.0110	0.0069
0.5	200	10.17	12	0.008	0.0299	0.0183
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	8.05				
2	50	9.29				
1	100	10.2	10	submerged	submerged	submerged
0.5	200	11.13	12	0.319	0.8374	0.7214

Table 5.7b. Overtopping Analysis of Little River South Floodwall (in meters).

		Overtopping Analysis of Flood Wall				
		Little River Option South Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	7.09	0.00	0.00	0.00	0.00
2	50	8.33	0.00	0.00	0.00	0.00
1	100	9.24	10.00	1.58	1.02	0.64
0.5	200	10.17	12.00	0.74	2.78	1.70
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	8.05	0.00	0.00	0.00	0.00
2	50	9.29	0.00	0.00	0.00	0.00
1	100	10.2	10.00	submerged	submerged	submerged
0.5	200	11.13	12.00	29.64	77.80	67.02

Table 5.8a. Overtopping Analysis of Little River Surge Barrier (in feet).

		Overtopping Analysis of Flood Wall				
		Little River Option South Barrier				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	7.09	10.7	0.011	0.2325	0.1557
2	50	8.33	13.2	0.009	0.3028	0.1995
1	100	9.24	15.2	0.015	0.3136	0.2441
0.5	200	10.17	17.7	0.013	0.3973	0.2529
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	8.05	10.7	0.049	0.5268	0.3924
2	50	9.29	13.2	0.041	0.5691	0.4070
1	100	10.2	15.2	0.042	0.6320	0.4422
0.5	200	11.13	17.7	0.033	0.6179	0.4166

Table 5.8b. Overtopping Analysis of Little River Surge Barrier (in meters).

		Overtopping Analysis of Flood Wall				
		Little River Option South Barrier				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	7.09	10.70	1.02	21.60	14.47
2	50	8.33	13.20	0.84	28.13	18.53
1	100	9.24	15.20	1.39	29.13	22.68
0.5	200	10.17	17.70	1.21	36.91	23.50
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	8.05	10.70	4.55	48.94	36.46
2	50	9.29	13.20	3.81	52.87	37.81
1	100	10.2	15.20	3.90	58.71	41.08
0.5	200	11.13	17.70	3.07	57.40	38.70

Table 5.9a. Overtopping Analysis of Little River North Floodwall (in feet).

		Overtopping Analysis of Flood Wall				
		Little River Option North Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	7.09				
2	50	8.33	9.4	0.005	0.0105	0.0062
1	100	9.24	10.9	0.017	0.0695	0.0463
0.5	200	10.17	12.9	0.014	0.0959	0.0616
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	8.05				
2	50	9.29	9.4	0.562	1.3131	1.2792
1	100	10.2	10.9	0.494	1.2269	1.0966
0.5	200	11.13	12.9	0.268	0.8694	0.7024

Table 5.9b. Overtopping Analysis of Little River North Floodwall (in meters).

		Overtopping Analysis of Flood Wall				
		Little River Option North Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	7.09	0.00	0.00	0.00	0.00
2	50	8.33	9.40	0.46	0.98	0.58
1	100	9.24	10.90	1.58	6.46	4.30
0.5	200	10.17	12.90	1.30	8.91	5.72
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	8.05	0.00	0.00	0.00	0.00
2	50	9.29	9.40	52.21	121.99	118.84
1	100	10.2	10.90	45.89	113.98	101.88
0.5	200	11.13	12.90	24.90	80.77	65.26

Table 5.10a. Overtopping Analysis of Biscayne South Floodwall (in feet).

		Overtopping Analysis of Flood Wall				
		Biscayne Option South Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	7.18				
2	50	8.53	9.1	0.001	0.0019	0.0011
1	100	9.48	10.6	0.010	0.0609	0.0423
0.5	200	10.41	12.6	0.007	0.0824	0.0538
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	8.14				
2	50	9.49	9.1	submerged	submerged	submerged
1	100	10.44	10.6	0.729	1.7466	1.6937
0.5	200	11.37	12.6	0.281	0.9978	0.8425

Table 5.10b. Overtopping Analysis of Biscayne South Floodwall (in meters).

		Overtopping Analysis of Flood Wall				
		Biscayne Option South Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	7.18	0.00	0.00	0.00	0.00
2	50	8.53	9.10	0.09	0.18	0.10
1	100	9.48	10.60	0.93	5.66	3.93
0.5	200	10.41	12.60	0.65	7.66	5.00
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	8.14	0.00	0.00	0.00	0.00
2	50	9.49	9.10	submerged	submerged	submerged
1	100	10.44	10.60	67.73	162.26	157.35
0.5	200	11.37	12.60	26.11	92.70	78.27

Table 5.11a. Overtopping Analysis of Biscayne Surge Barrier (in feet).

		Overtopping Analysis of Flood Wall				
		Biscayne Option Surge Barrier				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	7.18	9.15	0.015	0.2967	0.2188
2	50	8.53	12.15	0.019	0.2590	0.1750
1	100	9.48	13.15	0.019	0.4124	0.2895
0.5	200	10.41	14.65	0.020	0.3990	0.2741
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	8.14	9.15	0.215	0.9260	0.7921
2	50	9.49	12.15	0.520	0.5747	0.4309
1	100	10.44	13.15	0.096	0.8389	0.6459
0.5	200	11.37	14.65	0.069	0.7652	0.5724

Table 5.11b. Overtopping Analysis of Biscayne Surge Barrier (in meters).

		Overtopping Analysis of Flood Wall				
		Biscayne Option Surge Barrier				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	7.18	9.15	1.39	27.56	20.33
2	50	8.53	12.15	1.77	24.06	16.26
1	100	9.48	13.15	1.77	38.31	26.90
0.5	200	10.41	14.65	1.86	37.07	25.46
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	8.14	9.15	19.97	86.03	73.59
2	50	9.49	12.15	48.31	53.39	40.03
1	100	10.44	13.15	8.92	77.94	60.01
0.5	200	11.37	14.65	6.41	71.09	53.18

Table 5.12a. Overtopping Analysis of Biscayne North Floodwall (in feet).

		Overtopping Analysis of Flood Wall				
		Biscayne Option North Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	7.18	8.1	0.002	0.0197	0.0127
2	50	8.53	10.1	0.016	0.1426	0.1018
1	100	9.48	11.6	0.016	0.1826	0.1282
0.5	200	10.41	13.1	0.015	0.2225	0.1541
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (ft3/s/ft)	EurOtop Deterministic (ft3/s/ft)	EurOtop Probabilistic (ft3/s/ft)
5	20	8.14	8.1	submerged	submerged	submerged
2	50	9.49	10.1	0.200	0.6901	0.6056
1	100	10.44	11.6	0.132	0.6225	0.5131
0.5	200	11.37	13.1	0.099	0.6069	0.4794

Table 5.12b. Overtopping Analysis of Biscayne North Floodwall (in meters).

		Overtopping Analysis of Flood Wall				
		Biscayne Option North Wall				
		2079 w/high USACE SLR				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	7.18	8.10	0.19	1.83	1.18
2	50	8.53	10.10	1.49	13.25	9.46
1	100	9.48	11.60	1.49	16.96	11.91
0.5	200	10.41	13.10	1.39	20.67	14.32
		2079 w/high NOAA SLR for adaptability				
% AEP	Annual Recurrence Interval (yr)	SFLSSS Water Levels (ft)	Height of Wall (ft)	ACES Results (liters/s/m)	EurOtop Deterministic (liters/s/m)	EurOtop Probabilistic (liters/s/m)
5	20	8.14	8.10	submerged	submerged	submerged
2	50	9.49	10.10	18.58	64.11	56.26
1	100	10.44	11.60	12.26	57.83	47.67
0.5	200	11.37	13.10	9.20	56.38	44.54

5.3Allowable Overtopping Rates and Adaptability

The TSP for the Miami-Dade County Back Bay CSRM is based on the design of wall heights for the end of the period of economic analysis or end of the 50 year project life. To place the overtopping results into context, Table VI-5-6 from the CEM (**Table 5.2**) and Table 3.5 from the EurOtop – Overtopping Manual (**Table 5.3** in this report) was used. Based on **Table 5.2** and **Table 5.3**, the overtopping rates for the floodwall, at the design water level of year 2079 using the 90% CL and including SLR from the high USACE curve for all areas, were designed for the "no damage" condition for the heights analyzed in **Table 5.1** to **Table 5.12**. The allowable overtopping rate limit chosen was q = 2 litres/second per meter (0.0215 cubic feet/second per foot). There is no indication that overtopping will be a problem throughout the life of the project. An additional 50 years after the project life (year 2130), with the increase of SLR, the ACES results show damage could possibly occur if additional protection is not added to the crest or additional scour protection on the land side and the EurOtop method results show no damage if crest and rear slope are well protected. This why having floodwalls resilient to overtopping and adaptable to climate change can reduce the risks associated with overtopping events for some flood protection projects. Increasing the height of the structures after the end of the project economic period of analysis will need to be considered and analyzed to prevent damage from overtopping after the project economic period of analysis. The EurOtop method produced slightly higher overtopping rates for the walls.

Overtopping rates are typically included in the H&H analysis to account for the additional volume of water added to the interior storage, which could potentially increase the pump station capacity. Refer to **CHAPTER 7** of this sub-appendix for a detailed discussion on the H&H analysis.

Table 5.2 Allowable Overtopping Rates for ACES values (taken from CEM table VI-5-6).

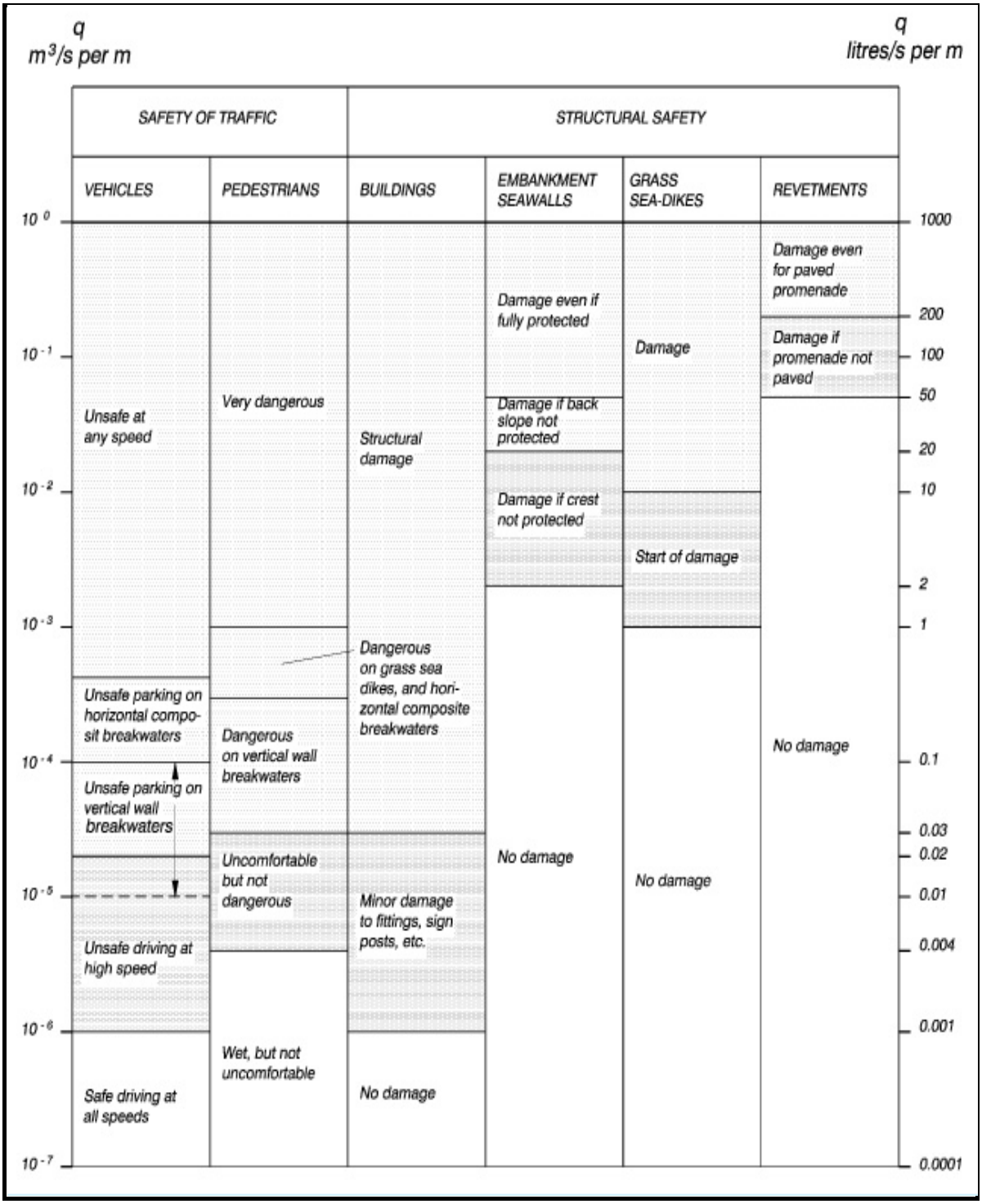


Table 5.3. Allowable Overtopping Rates for EurOtop values (EurOtop Manual Table 3.5).

Hazard type and reason	Mean discharge q (l/s/m)
Embankment seawalls/sea dikes	
No damage if crest and rear slope are well protected	50–200
No damage to crest and rear face of grass covered embankment of clay	1–10
No damage to crest and rear face of embankment if not protected	0.1
Promenade or revetment seawalls	
Damage to paved or armoured promenade behind seawall	200
Damage to grassed or lightly protected promenade or reclamation cover	50

CHAPTER 6 Wave Forces on a Vertical Wall

6.1 Calculation of Wave Forces on a Vertical Wall

A characteristic of vertical wall is that the kinetic energy of the wave is stopped suddenly at the wall face. The energy is then reflected or translated by vertical motion of the water along the wall face. The upward component of this can result in the wave crests to rise and/or to double their deep water height (non-breaking case). The downward component causes very high velocities at the base of the wall and horizontally away from the wall for ½ of a wavelength, thus causing erosion and scour. The forces exerted on vertical walls by reflected water waves were calculated for the vertical flood wall proposed for this study. There are two methods that are typically used for calculating wave forces: the Goda method and the Minikin method. The method used for this study to calculate the wave forces on a vertical wall, was Goda method. The Goda method was used because the equations for this method are applicable to nonbreaking and breaking (no distinctions are made between the two), which the Minikin method does not. The formulas give additional force due to the waves to the Stillwater hydrostatic force, which adds to the total forces on the wall. The Goda method assumes trapezoidal shape for pressure distribution along front of a vertical (see **Figure 6.1**). The pressure at the top of the wall (labeled as P_2), Stillwater surface elevation (labeled as P_1), and toe of wall structure are the forces calculated (P_3).

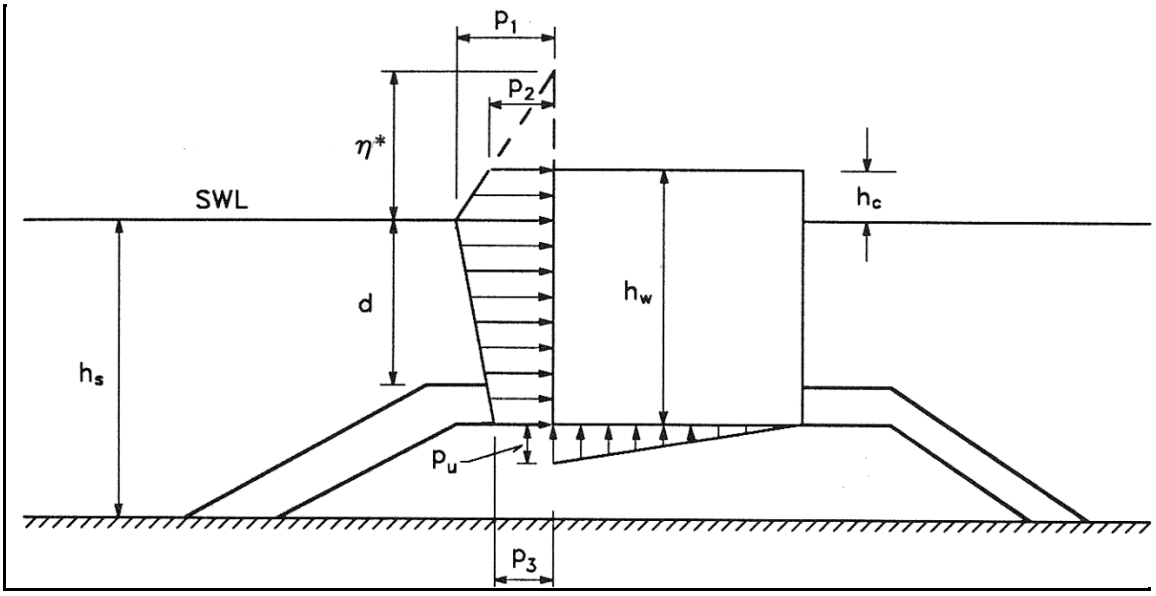


Figure 6.1. Trapezoidal Shape for Pressure Distribution and Definition sketch for Goda formula for wave induced pressure under wave crest on a vertical wall.

The following is an explanation of the Goda Method's equations and input parameters used to calculate wave forces on a vertical wall:

- Elevation to which wave pressure is exerted is calculated by the following equation:

  $\eta* = 0.75(1+ \cos\beta)H_{design}$

  β = direction of waves with respect to breakwater normal (for waves approaching normal to breakwater, $\beta = 0$).

For the Goda method, a maximum wave height is calculated from the significant wave height ($H_{design} = 1.8H_s$). For the calculation of the wave forces, the SFLSSS calculated waves presented in **Table 4.1** were used for the correlating SWL.

- Effect of wave period on pressure distribution ($\alpha 1$):

  $\alpha 1 = 0.6 + 0.5 \left(\frac{2khs}{sinh2khs}\right)^2$

  minimum = 0.6 (deep water), maximum = 1.1 (shallow)

- Increase in wave pressure due to shallow mound ($\alpha 2$):

$\alpha 2$ = minimum of $\frac{hb-d}{3hb}\left(\frac{Hdesign}{d}\right)^2$ or $\frac{2d}{Hdesign}$

- Linear pressure distribution ($\alpha 3$):

  $\alpha 3 = 1 - \frac{hw-hc}{3hs}\left\{1 - \frac{1}{d(cosh)khs}\right\}^2$

- Pressure on Front of Vertical Wall:

$$p_1 = 0.5(1+\cos\beta)(\alpha_1 + \alpha_* \cos^2\beta)\gamma H_{design}$$

$$p_2 = \begin{cases} \left(1 - \frac{h_c}{\eta^*}\right)p_1 & \text{for } \eta^* > h_c \\ 0 & \text{for } \eta^* \le h_c \end{cases}$$

$$p_3 = \alpha_3 p_1$$

- Wave Force (Total Horizontal Force to breaking wave):

  $F = 0.5\ (P_1 + P_3)h_c + 0.5(P_1 + P_2)(h_w - h_c)$

- Overturning Moment: $M = F \times h_c$

The Goda forces for the wall heights were calculated in a Microsoft Excel spreadsheet. **Table 6.1 to Table 6.4** summarizes the wave forces for the different wall heights from **Table 5.1 to Table 5.12** calculated in **Section 5.2.3**, **Individual Model Results of Floodwall Overtopping Analysis**, for the AEPs of 5% flood (20 yr.), 2% flood (50 yr.), and 1% flood (100 yr.) SWL elevations for the year 2079. Note that for the calculation of wave forces on a vertical wall, an average ground elevation at each structure was determined using the elevations from the SFLSSS modeling grid. The wave forces matching the Top of Wall Elevation is highlighted in blue.

Table 6.1. Miami River Wave Forces calculated for different wall heights for AEPs 5% (20 year), 2% (50 year), 1% (100 year), and 0.5% (200 year).

		Wave Forces (lb/ft)				Overturning Moment (lb-ft/ft)			
	Top of Wall Elevation [ft NAVD88]	20-YR SWL	50-YR SWL	100-YR SWL	200-YR SWL	20-YR SWL	50-YR SWL	100-YR SWL	200-YR SWL
Miami River 1 South	7.9	9	146	328	619	2	-37	-232	-739
	10.9	72	438	759	1182	47	743	1303	1563
	13.4	126	700	1118	1651	135	1252	3193	5026
	16.4	190	1114	1552	2214	298	2495	4415	9303
Miami River 1 South Wall in Water	13.2	4631	6187	7216	8322	45950	58774	65237	70074
	16.2	4779	6405	7465	8590	53434	73402	84305	93707
	18.2	4878	6551	7630	8769	55779	81121	95349	108184
	20.7	5328	6732	7838	8993	61940	87687	106485	124049
Miami River 1 Surge Barrier	15.9	8938	11394	13047	14651	133198	165404	184311	199240
	18.9	9009	11561	13264	14897	148431	190570	216304	237987
	20.9	9056	11673	13409	15060	155092	204284	234906	261454
	23.4	9115	11813	13591	15265	158733	217205	254308	287341
Miami River 1 North	9.5	542	1033	1474	2033	1031	1959	2240	2017
	13.0	981	1603	2184	2877	2044	5677	8333	10573
	15.5	1359	2011	2690	3480	3355	7363	12671	17663
	18.5	1812	2654	3298	4203	5369	10079	16065	25515

Table 6.2. Edgewater Wave Forces calculated for different wall heights for AEPs 5% (20 year), 2% (50 year), 1% (100 year), and 0.5% (200 year).

		Wave Forces (lb/ft)				Overturning Moment (lb-ft/ft)			
	Top of Wall Elevation [ft NAVD88]	20-YR SWL	50-YR SWL	100-YR SWL	200-YR SWL	20-YR SWL	50-YR SWL	100-YR SWL	200-YR SWL
Edgewater (land)	9.1	547	908	1258	1765	1004	1834	2283	2464
	11.6	859	1303	1749	2368	1597	3929	5771	7635
	13.6	1171	1619	2142	2851	2512	5002	8469	12299
	16.1	1561	2126	2634	3454	3958	6712	10700	17792
Edgewater (water)	11.4	3078	4145	4934	5782	24146	31382	35712	38722
	13.9	3198	4331	5159	6024	28305	39466	46589	52567
	15.4	3270	4443	5294	6170	29492	43294	52295	60284
	17.4	3572	4592	5474	6364	32588	46787	58528	69466

Table 6.3. Little River Wave Forces calculated for different wall heights for AEPs 5% (20 year), 2% (50 year), 1% (100 year), and 0.5% (200 year).

		Wave Forces (lb/ft)				Overturning Moment (lb-ft/ft)			
	Top of Wall Elevation [ft NAVD88]	20-YR SWL	50-YR SWL	100-YR SWL	200-YR SWL	20-YR SWL	50-YR SWL	100-YR SWL	200-YR SWL
Little River (south)									
	10.0			77	205			51	145
	12.0			186	389			154	594
Little River (surge barrier)	10.7	2113	3044	3910	4923	12000	17109	21169	25318
	13.2	2322	3357	4326	5461	14457	23078	30083	37486
	15.2	2592	3608	4659	5891	16039	26553	36332	46809
	17.7	3111	3921	5075	6428	20212	28385	42150	57052
Little River (north)									
	9.4		127	282	475		108	283	371
	10.9		225	431	690		205	703	1234
	12.9		373	645	977		454	1048	2448

Table 6.4. Biscayne Canal Wave Forces calculated for different wall heights for AEPs 5% (20 year), 2% (50 year), 1% (100 year), and 0.5% (200 year).

		Wave Forces (lb/ft)				Overturning Moment (lb-ft/ft)			
	Top of Wall Elevation [ft NAVD88]	20-YR SWL	50-YR SWL	100-YR SWL	200-YR SWL	20-YR SWL	50-YR SWL	100-YR SWL	200-YR SWL
Biscayne (south)									
	9.1		29	117	243		12	45	16
	10.6		79	222	410		44	266	505
	12.6		147	381	632		130	489	1320
Biscayne (surge barrier)	9.2	1336	2299	2861	3379	7296	11158	12707	13479
	12.2	1322	2396	2997	3538	9144	17039	20864	23678
	13.2	1412	2428	3042	3590	9867	18487	23175	26774
	14.7	1597	2477	3110	3670	11405	19966	26043	30911
Biscayne (north)	8.1	127	322	481	676	118	266	216	25
	10.6	292	545	739	969	329	1158	1705	2080
	12.6	439	737	946	1203	632	1557	2823	3987
	14.6	586	1010	1170	1437	1036	2418	3391	5487

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 T-walls.

CHAPTER 7 Hydrologic and Hydraulic Analysis

The yearly rainfall average is approximately 61 inches. Miami-Dade County has a total area of approximately 2,431 square miles, of which 533 square miles (21.9%) is water. Miami-Dade County is a low-lying densely populated area, which is vulnerable to rising sea levels. The climate in Miami-Dade County is tropical monsoon climate with mean monthly temperatures above 64 degrees Fahrenheit. August is typically the warmest month (average high of 88.5 degrees Fahrenheit); January is typically the coldest month (average high of 73.8 degrees Fahrenheit). Southeastern Florida is characterized by distinct wet and dry seasons, high rainfall and evapotranspiration rates, relatively low topographic relief, and high water-table conditions.

Since the Miami-Dade County Back Bay CSRM TSP has determined storm surge barriers and floodwalls were needed to protect against coastal storms, the storm surge barriers and floodwalls may affect the rainfall runoff peak discharge relief that was naturally occurring. EM 1110-2-1413 references that if the storage, as a result of the tidal barriers, rises behind the level of protection to a level beyond what has occurred naturally, then a relief system will be recommended. That relief system could be a combined collector drain and/or pump station(s) or other suggestions depending on the circumstances. Therefore, in the PED phase, the Miami-Dade County Back Bay CSRM several relief options including pump stations will be evaluated to relieve interior drainage. The detailed hydrologic analysis will include the 24 hour point rainfall peak estimates for the 2, 5, 10, 25, 50, 100, and 500 year frequency events. These storm frequencies are part of the hydrologic analysis to understand the effects of the floodwall on the protected area behind the floodwall. This H&H analysis is recommended to include protective systems currently in place and/or designed to understand the effect of the proposed structural measures.

The current structural measures proposed are to be built across the Central and Southern Flood control system. Coordination with the South Florida Water Management District will be essential for any future phase of this Miami-Dade County Back Bay CSRM project.

To have an understanding of the magnitude of historical storms that have occurred in the Miami-Dade County area, see **Table 1.1** which shows the high tide level storm events that have occurred in the area along with the rainfall amounts associated with the event. Some tidal events do not have large amounts of rain accompanying the storm.

In the 1950s, the USACE built canals to manage the surface water. Westward expansion has been allowed for urban and agricultural activities. The canals control urban flooding, control seawater intrusion, and supplies recharge to the production well fields. The South Florida Water Management District (SFWMD) operates the canal structures in all of the canals in the focus areas and in others in South Florida. In the PED phase it is highly recommended to coordinate the proposed storm surge barriers and floodwalls with the SFWMD control structures.

For the Miami-Dade County Back Bay CSRM, a simplified Hydrologic model using the USACE Hydrologic Engineering Center (HEC) Hydrologic Modeling System (HMS) software version 4.3, was used to produce a hyetograph (excess rainfall) to use in the USACE HEC software, River Analysis System (HEC-RAS) version 5.0.7. **Figure 7.1** shows the four hydrologic areas for the structural solutions. A two dimensional (2D) approach was done to check against an existing report provided – CH2M Report "Surge and Flood Modeling for Miami-Dade County (Task 2.10) in June 2015. In the CH2M report a 2D hydraulic computer model was developed using 72 hour precipitation data. That same 72 hour precipitation data was used in HMS. The NOAA Atlas 14 Precipitation Frequency Data Server (PFDS) was also used for the 1% AEP (100 year) rainfall for a duration of four days. Both precipitations were used in a 1 sub-basin (composing of the total area of the four focus areas) model and another of the entire urban areas of the County west of Biscayne Bay. Both HMS sub-basins yielded the same rainfall hyetograph as expected. The same loss parameters were used in both HEC-HMS models. See **Figure 7.2** for comparable results. HEC-RAS was then used to create a 2D model incorporating the "with precipitation" and "without precipitation" conditions for both "with the floodwall" and for the "without the project" conditions.

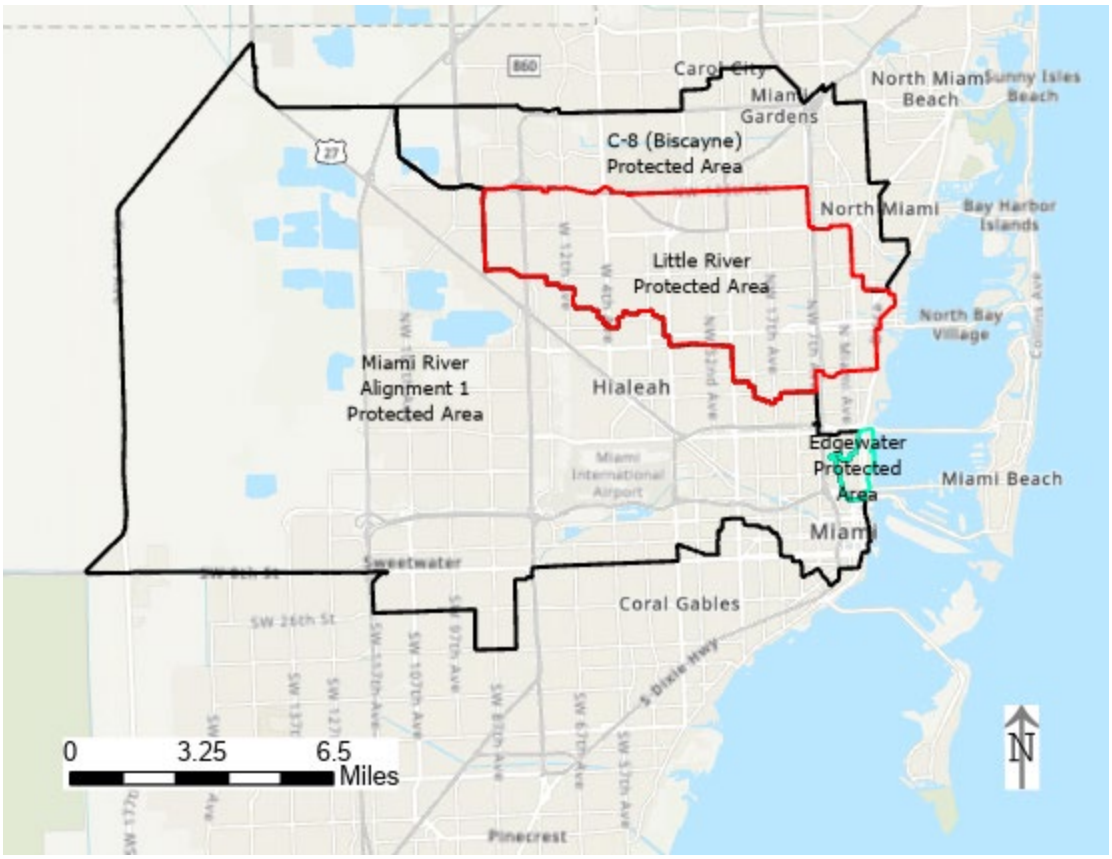
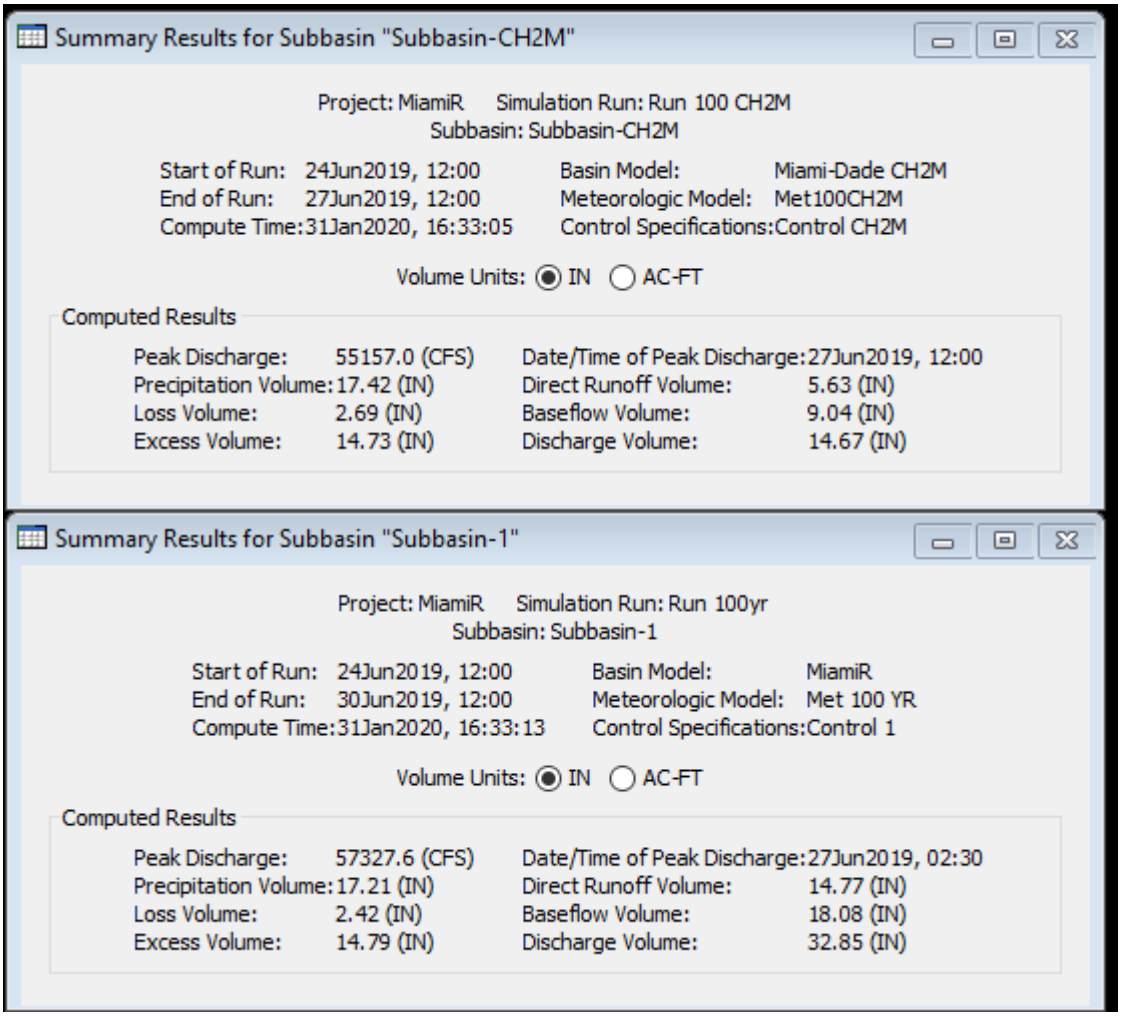


Figure 7.1. Image of the four hydrologic areas with proposed projects.



Summary Results for Subbasin "Subbasin-CH2M"

Project: MiamiR Simulation Run: Run 100 CH2M
Subbasin: Subbasin-CH2M

Start of Run:	24Jun2019, 12:00	Basin Model:	Miami-Dade CH2M
End of Run:	27Jun2019, 12:00	Meteorologic Model:	Met100CH2M
Compute Time:	31Jan2020, 16:33:05	Control Specifications:	Control CH2M

Volume Units: (•) IN ( ) AC-FT

Computed Results

Peak Discharge:	55157.0 (CFS)	Date/Time of Peak Discharge:	27Jun2019, 12:00
Precipitation Volume:	17.42 (IN)	Direct Runoff Volume:	5.63 (IN)
Loss Volume:	2.69 (IN)	Baseflow Volume:	9.04 (IN)
Excess Volume:	14.73 (IN)	Discharge Volume:	14.67 (IN)

Summary Results for Subbasin "Subbasin-1"

Project: MiamiR Simulation Run: Run 100yr
Subbasin: Subbasin-1

Start of Run:	24Jun2019, 12:00	Basin Model:	MiamiR
End of Run:	30Jun2019, 12:00	Meteorologic Model:	Met 100 YR
Compute Time:	31Jan2020, 16:33:13	Control Specifications:	Control 1

Volume Units: (•) IN ( ) AC-FT

Computed Results

Peak Discharge:	57327.6 (CFS)	Date/Time of Peak Discharge:	27Jun2019, 02:30
Precipitation Volume:	17.21 (IN)	Direct Runoff Volume:	14.77 (IN)
Loss Volume:	2.42 (IN)	Baseflow Volume:	18.08 (IN)
Excess Volume:	14.79 (IN)	Discharge Volume:	32.85 (IN)

Figure 7.2. Comparing HMS rainfall.

The results illustrate the peak discharges are similar for both sub-basin models used. Therefore, the 100 year frequency rainfall from NOAA was used in HEC-RAS. Precipitation in HEC-RAS is applied equally to all cells within the 2D flow area. Also, the precipitation option is used to apply rainfall excess (Rainfall minus losses due to interception/infiltration) directly to a 2D flow area. The results of the precipitation added to the HEC-RAS model may appear in blocks as that is the result of the method used in HEC-RAS in calculating equally to all cells. These rainfall result will be refined before the Final report.

7.1Hydrologic and Hydraulic Parameters

The hydrologic parameters used in the HEC-HMS model were guided by several reports. One of the reports was the Scientific Investigations Report (SIR) 2014-5162 Version 1.2, July 2016 completed by the United States Geological Survey (USGS). The soils group in the County ranges from well drained (high filtration rate and low runoff potential) in the western portions to poorly drained (moderate infiltration to very slow infiltration and higher runoff potential) in the eastern portions. In the western portions, the soils are classified as The Everglades and Sandy Flatlands. Towards the eastern areas there is an Atlantic Coastal Ridge and Coastal Marshes and Mangrove Swamps.

For the parameters used in HEC-HMS, a Time of concentration, baseflow, percent impervious, and losses were estimated base on referencing the USGS SIR 2016 report, the Canals in South Florida: A Technical Support Document, Appendices A to C, and the Flood Protection Level of Service (LOS) Analysis for the C-4 Watershed. Clark's T_c (time of concentration) and R (storage coefficient) were utilized in the analysis. R, which represents a storage relationship of the hydrograph, is typically one half of T_c, but can be smaller in steep, mountainous areas or greater (as high as equal to T_c) in flat, swampy areas. In the case of Miami-Dade County, R was increased to 0.9 of T_c to represent the flatness of the watershed and the storage associated with ponding and overland flow (down the streets and across lawns) when the capacity of the drainage pipes are exceeded. The storm parameters assumed in HEC-HMS was a 50 percent intensity position, 15 minute intensity duration, and a storm duration of 4 days. The HEC-HMS models assembled for this 10% design ran for 6 days to see the effects of the full runoff hydrographs. The roughness coefficients (Manning's n values) were estimated from a Miami Land Use grid.

Modeling the coastal tidal storm surge with SLR for the 1% AEP in the year 2079 indicated the need for a barrier system to limit or prevent the storm surge from a coastal storm. The HEC-RAS results indicated that by expanding the 2D flow area to include the entire urban area in Miami-Dade County there is a hydraulic connection from the area to the north of Arch Creek which is the Aventura area. Suggest a pump station be considered for the Arch Creek area to relieve the inflow from the Aventura area. No analysis prior to this

Draft Sub Appendix has been done to determine the size of a pump station needed to relieve the Arch Creek area. There are two existing pump stations in the Arch Creek area that could possibly be upgraded to handle the additional volume of surge coming from the Aventura area. Also, by expanding the 2D flow area to include the entire urban area in Miami-Dade County, a few hydraulic connections were noted. The entry from the Coral Gables Water Way west of the high ridge near Cartagena Park indicates connections in the interior with the watersheds to the north and south. The entry point south of Snapper Creek Lakes Club on Old Cutler Rd.at the canal indicates an entry point where another hydraulic connection leads to connections north of this watershed. Not included in this Draft report are the measures considered to address these hydraulic connections. Recommendations and/or possible other measures will be discussed before the Final Report.

The initial conditions in HEC-RAS for the boundary conditions were in NAVD88 and chosen to be 1.5 feet, which is estimated to be the mean value of the predicted tidal range in 2079. The diurnal tide range is 3 feet. See **Figure 7.3** for a typical range of the Virginia Key, Biscayne Bay, FL NOAA gauge (#8723214).

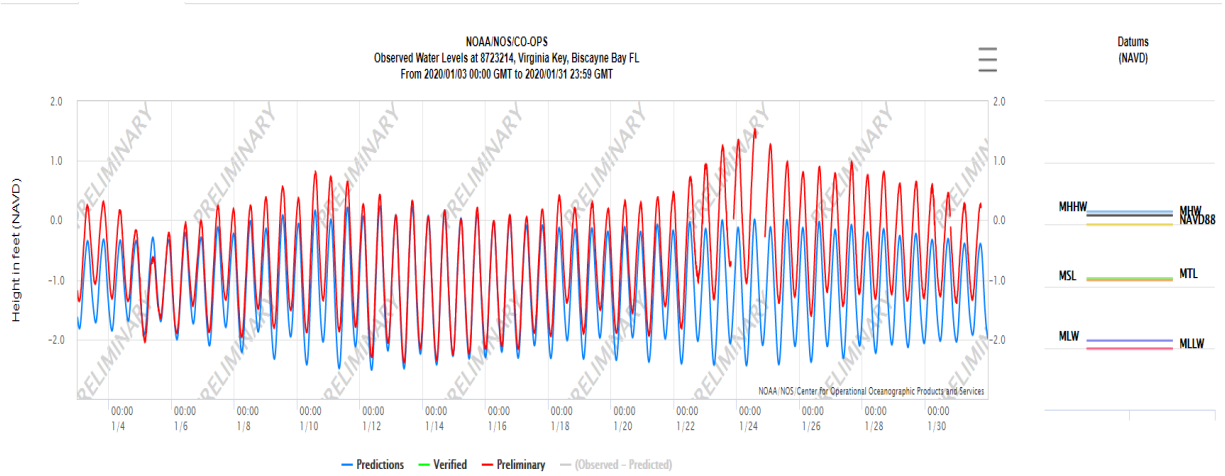


Figure 7.3. Predicted range of tide at the Virginia Key, Biscayne Bay, FL NOAA gauge

The downstream boundary condition used in HEC-RAS was a stage hydrograph and based off of the stage hydrograph during Hurricane Irma in September 2017 at the Virginia Key gauge. See **Figure 7.4** for the stage hydrograph during Hurricane Irma at the Virginia Key, Biscayne Bay, FL gauge. The 2017 Irma hydrograph was scaled up to reach the 2079 peak water levels for the 1% AEP (100 year) (See **Figure 7.5**). Each focus area had a

different 2079 peak water level as shown in **Table 3.3.** For each of the nine boundary conditions (See **Figure 7.6**) and corresponding peak water levels there was a scaled hydrograph for the 2079 condition. The use of year 2079 water levels is synonymous with the Future With Project (FWP) or Future Without Project (FWOP) conditions.

The boundary conditions used represent the storm surge approach from the bay towards the west. The save points chosen represent the some of the higher SWLs in the region closest to the shore. The Final Report will include the areas east of the current boundary conditions. A different HEC-RAS model with a 2D flow area will be developed and use the boundary conditions from the Bay side for the west boundary and save points in the Atlantic side will be used for the east boundary condition.

The Hurricane Irma storm was chosen to replicate a coastal storm since the peak tide elevation for the hurricane was the highest recorded tide level at the Virginia Key gauge (See **Table 1.1**) since 1994.

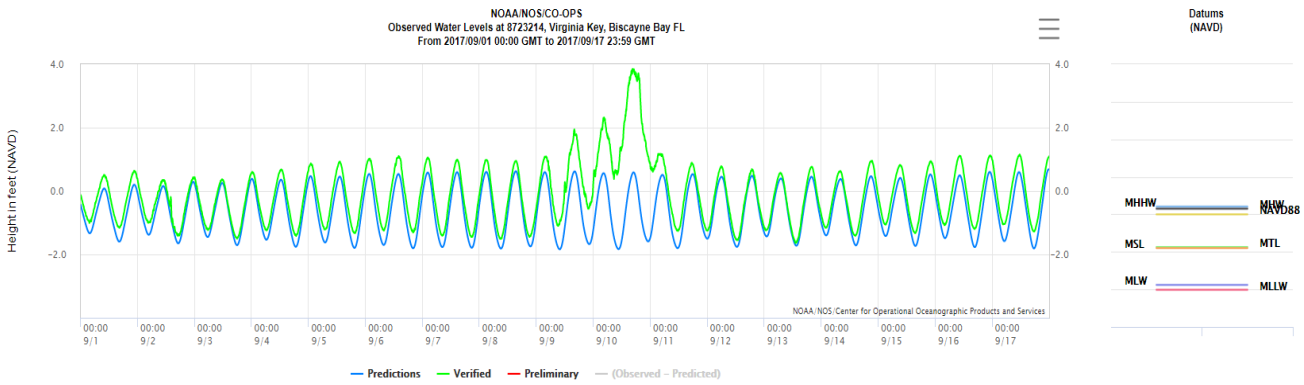


Figure 7.4. 2017 Hurricane Irma stage hydrograph.

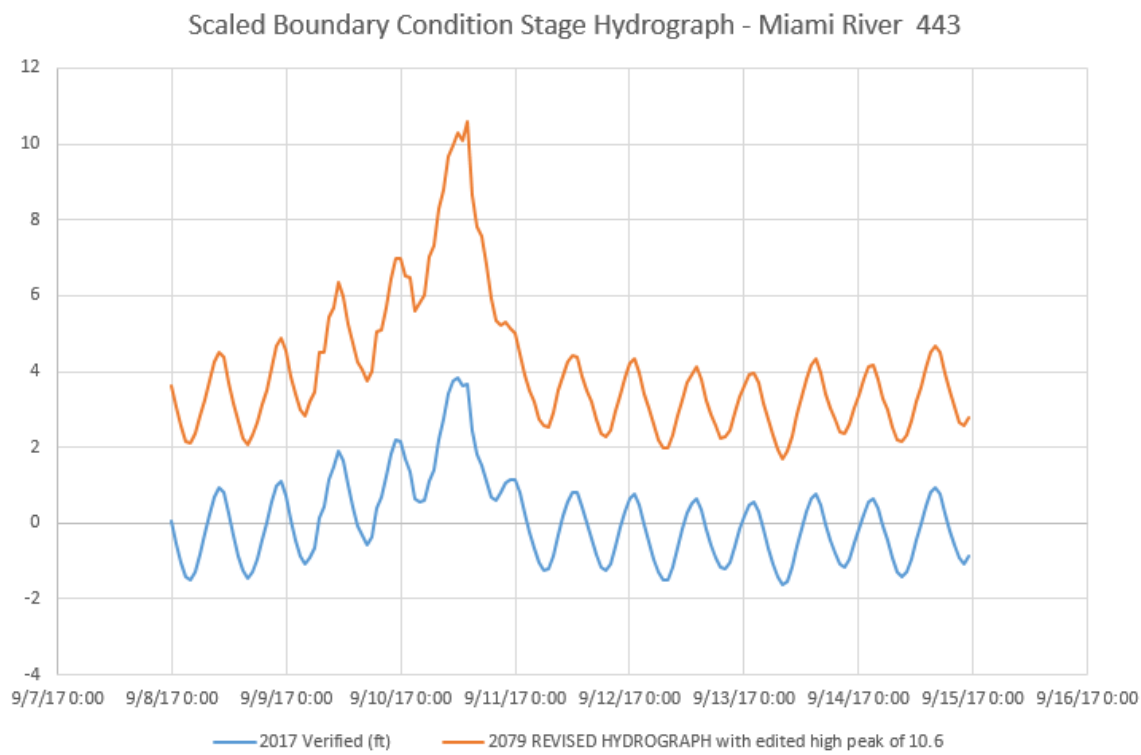


Figure 7.5. Boundary Condition Scaled up Stage Hydrograph for Miami River Save Point 443.

The Hurricane Irma in 2017 hydrograph was used in HEC-RAS for the "without project and with precipitation" and the "without project and without precipitation" conditions. They were also used for the "with-project" conditions as well. Rainfall was used in HEC-RAS as a method to compare to the CH2M results.

The total SLR in the year 2079 from the year 2018 is 3.29 feet NAVD88 using the high USACE SLR curve. The total SLR in the year 2079 from the year 2018 is 4.25 feet NAVD88 using the NOAA high curve. See **Table 7.1** for the values. In the CH2M hydraulic model the condition modeled was the SLR of 4 feet with and without 100 year rainfall.

Table 7.1. Total SLR adjustment at the Vaca Key NOAA tide gauge.

VACA KEY	USACE Low Curve		USACE Intermediate Curve		USACE High Curve		NOAA High Curve	
SLR Adjustments	[ft]	[m]	[ft]	[m]	[ft]	[m]	[ft]	[m]
1992 to 2018	0.310	0.095	0.370	0.113	0.560	0.171	0.660	0.201
2018 to 2030	0.150	0.046	0.220	0.067	0.430	0.131	0.530	0.162
2030 to 2079	0.590	0.180	1.130	0.345	2.860	0.872	3.720	1.134
2018 to 2079	0.740	0.226	1.350	0.412	3.290	1.003	4.250	1.296

Several scenarios in HEC-RAS were modeled. The scenarios are as follows:

1. Without Project and No Precipitation using the 2079 1% SWL including SLR for entire County west of Biscayne Bay
2. Without Project and With Precipitation using the 2079 1% SWL including SLR for entire County west of Biscayne Bay
3. With Project and No Precipitation using the 2079 1% SWL including SLR for entire County west of Biscayne Bay
4. With Project and With Precipitation using the 2079 1% SWL including SLR for entire County west of Biscayne Bay

A 2D flow area was drawn to include the entire county of Miami-Dade developed areas. And that 2D flow area contained over 2 million 100 feet by 100 feet cells. The 2D flow area cells were increased to 150 feet by 150 feet to increase the stability and speed of runtime. The lateral structures in the 2D flow area have 50 foot by 50 foot cells surrounding them. The lateral structures in the HEC-RAS software included the SFWMD structures S-25b, S-26, S-27, S-28, and G-58.

Tailwater Elevations Modeled in HEC-RAS		
Save point	Boundary Condition Name	1% 2079 SWL incl SLR
437	North Biscayne	9.19
621	Biscane Canal	9.48
614	Little River	9.24
572	Edgewater	9.48
443	Miami River	10.6
635	Coconut Grove	13.22
539	South Biscayne BC1	13.63
491	South Biscayne BC2	13.54
461	South Biscayne BC3	12.42

Figure 7.6. Boundary Conditions for the HEC-RAS 2D Flow Area for Miami-Dade County.

Figure 7.7 and **Figure 7.8** illustrate Scenarios 1 and 2. The resultant depth map produced has the following conditions: tailwater conditions included the 2079 AEP 1% (100 year) SWL including SLR. **Figure 7.9** and **Figure 7.10** illustrate Scenarios 3 and 4. The resultant depth maps produced has the following condition: tailwater conditions included the year 2079 AEP 1% (100 year) SWL including SLR. The HH&C Sub Appendix Attachment 2 includes additional depth maps at a larger scale.

Please note the following figures are subject to change as the HEC-RAS modeling will need to be recomputed to include the output design water level with the greatest net benefits from the economic modeling.

The inundation depth maps are showing the maximum depth. There are many points of interest in the 2D flow area that are interactive when illustrating the HEC-RAS results for depths and timing information. To capture the depth results at some points of interest, profile lines were created at the proposed floodwalls and surge barriers. Plots of the maximum depths of water in feet during the event modeled along the proposed structures are shown in **Figures 7.11 to 7.15.** The plots illustrate the difference in depths for the future with project (FWP) and future without project (FWOP) scenarios of the year 2079 1% AEP SWL elevations. The plots indicate a reduction of depth of water in feet in the inundation as expected. The plots of the scenarios with precipitation are not available as that analysis is under development. The depth plots with precipitation are to be included in the Final Report.

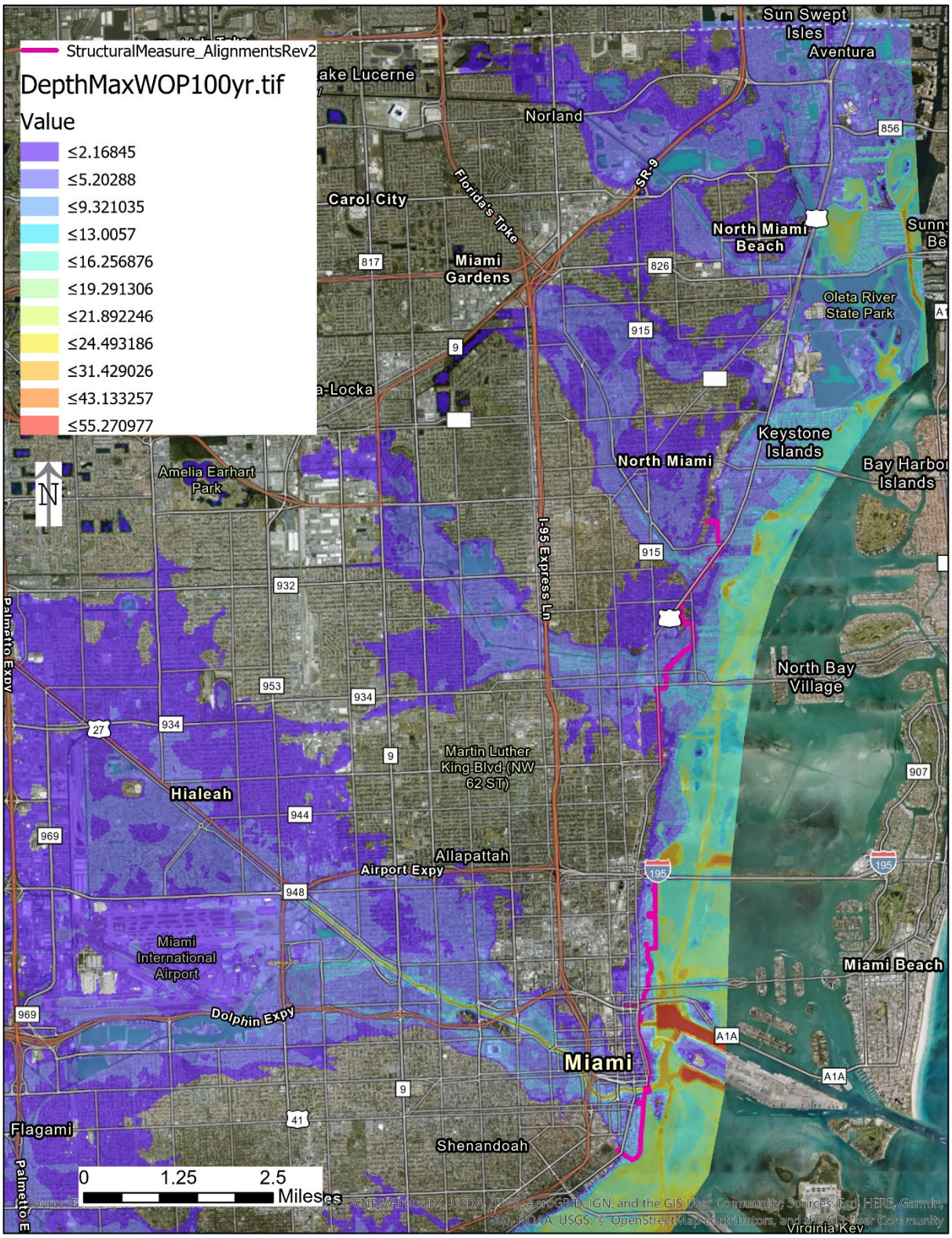


Figure 7.7. Depth (feet) Inundation layer from HEC-RAS for the Without Project and Without Precipitation at 1% 2079 SWL including SLR and Storm Surge. All focus areas - Scenario 1.

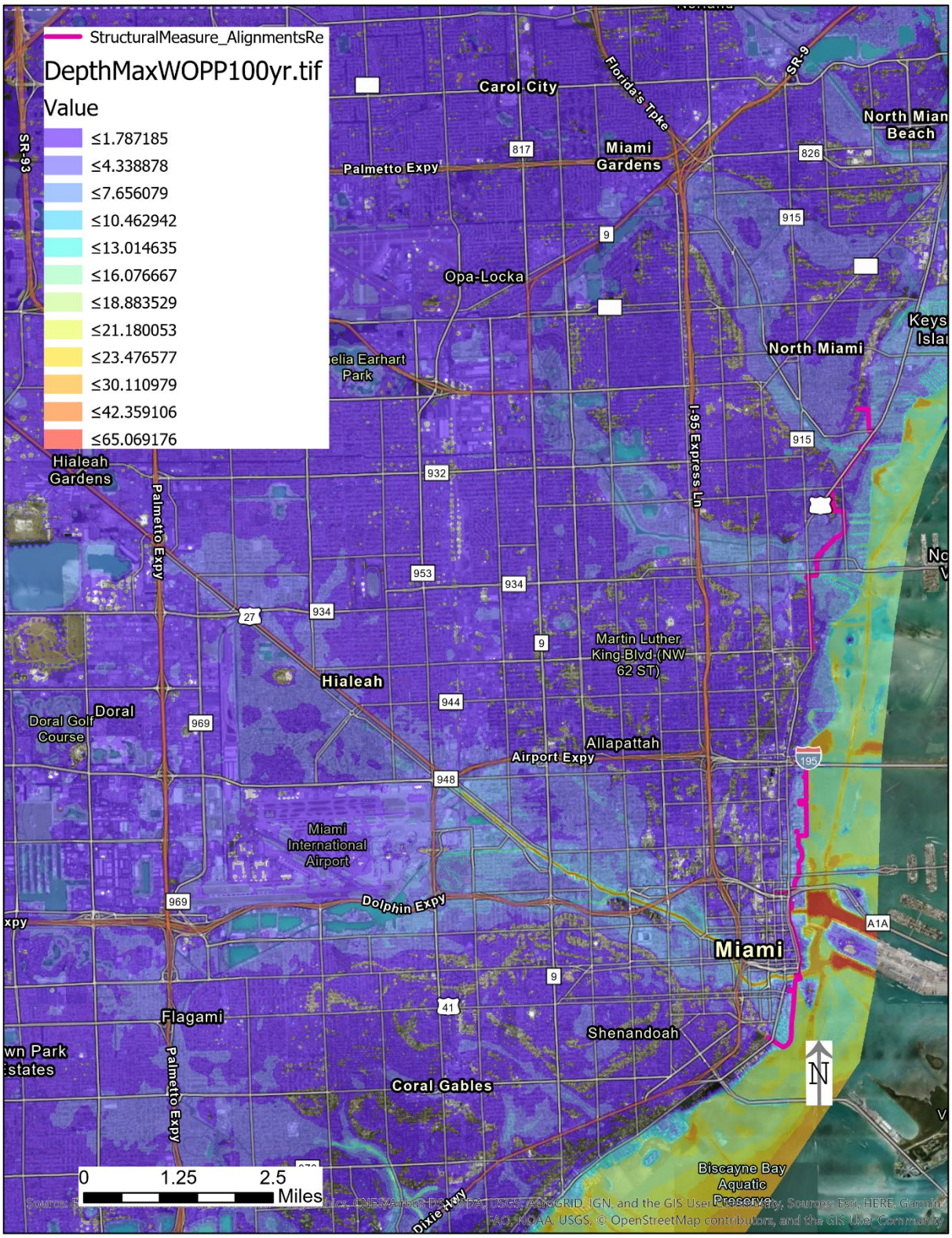


Figure 7.8. Depth (feet) Inundation layer in HEC-RAS for the Without Project and With Precipitation at 1% 2079 SWL including SLR and Storm Surge. All focus areas - Scenario 2.

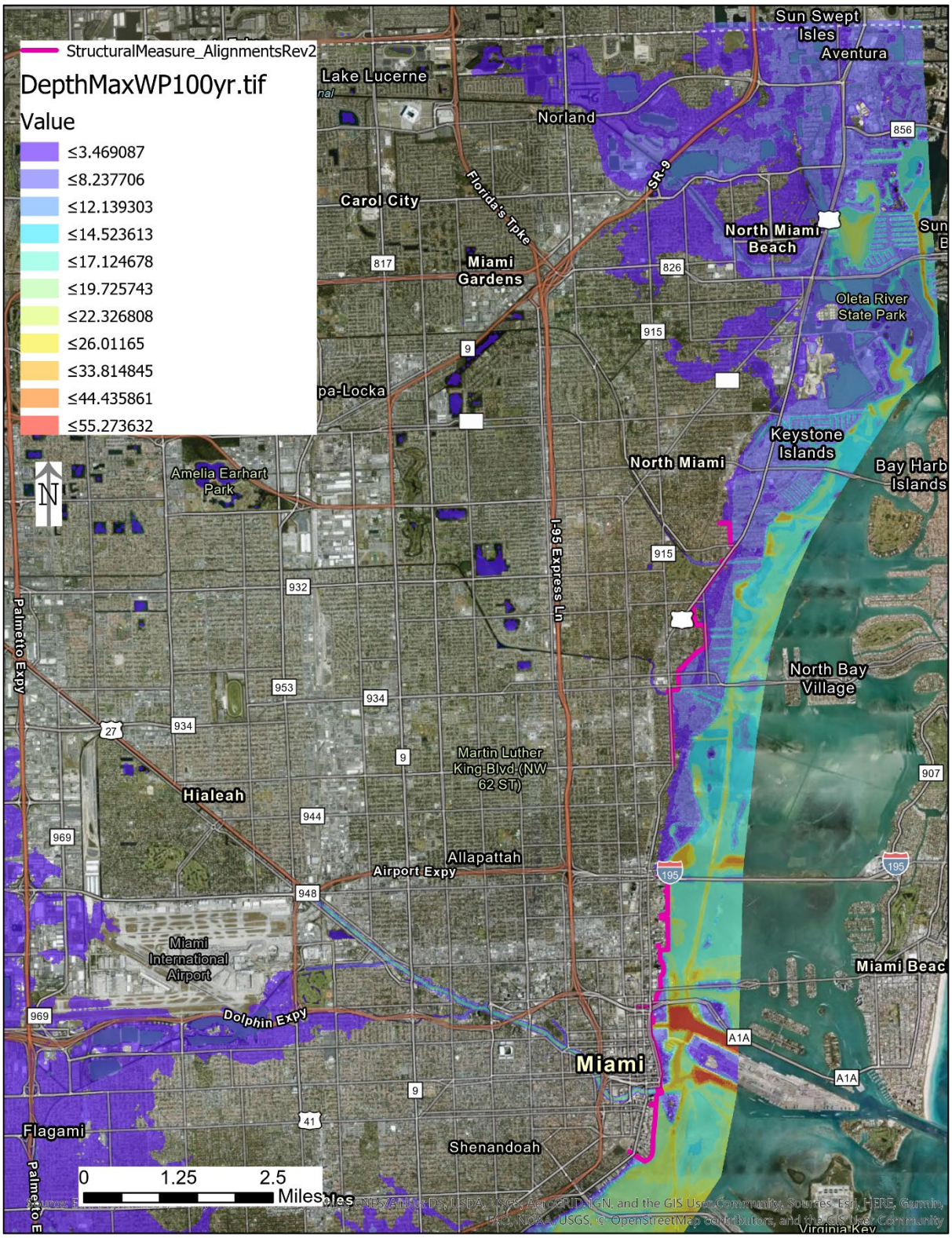


Figure 7.9. Depth (feet) Inundation layer in HEC-RAS for the With Project and Without Precipitation at 1% 2079 SWL including SLR and Storm Surge. All focus areas - Scenario 3.

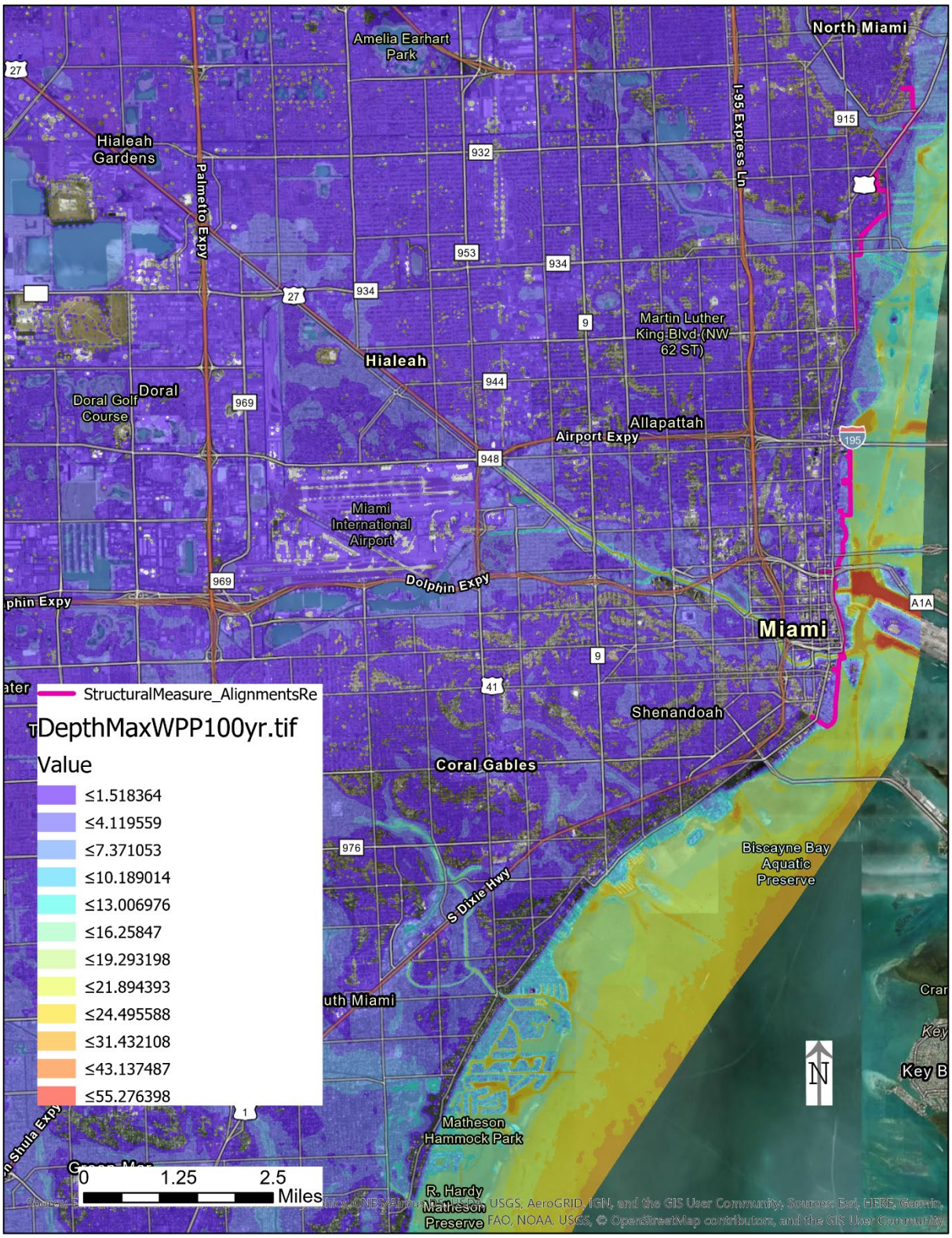


Figure 7.10. Depth (feet) Inundation layer in HEC-RAS for the With Project and With Precipitation at 1% 2079 SWL including SLR and Storm Surge. All focus areas - Scenario 4.

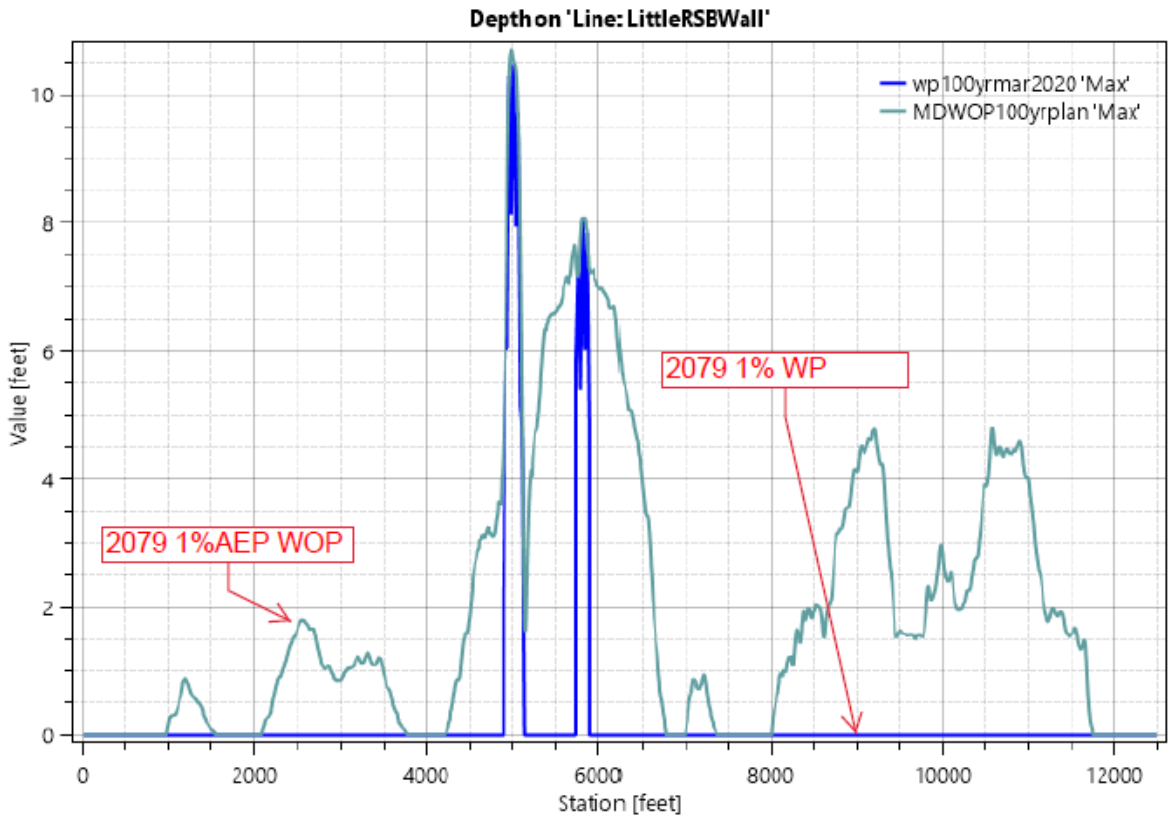


Figure 7.11. Depth profile plot along the Little River floodwall in HEC-RAS for the With Project and Without Project at 1% 2079 SWL elevation. Without Precipitation.

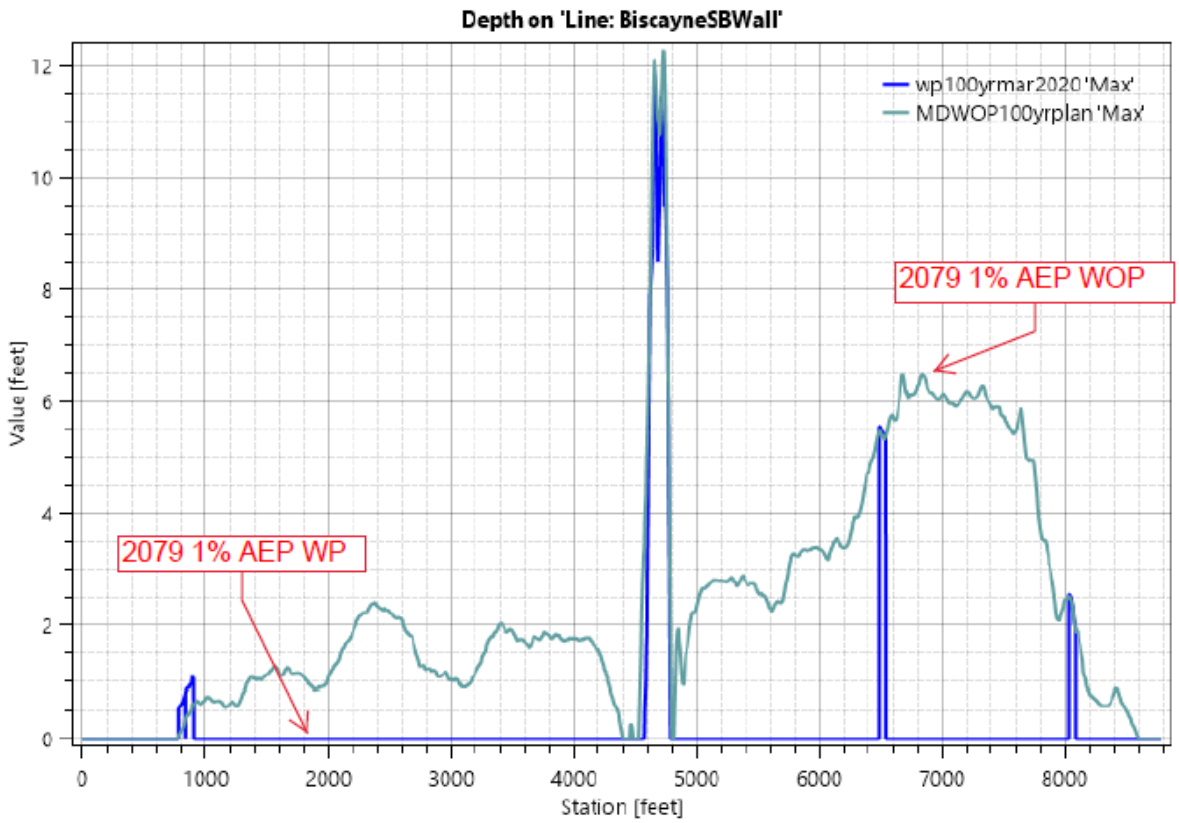


Figure 7.12. Depth profile plot along the Biscayne floodwall in HEC-RAS for the With Project and Without Project at 1% 2079 SWL elevation. Without Precipitation.

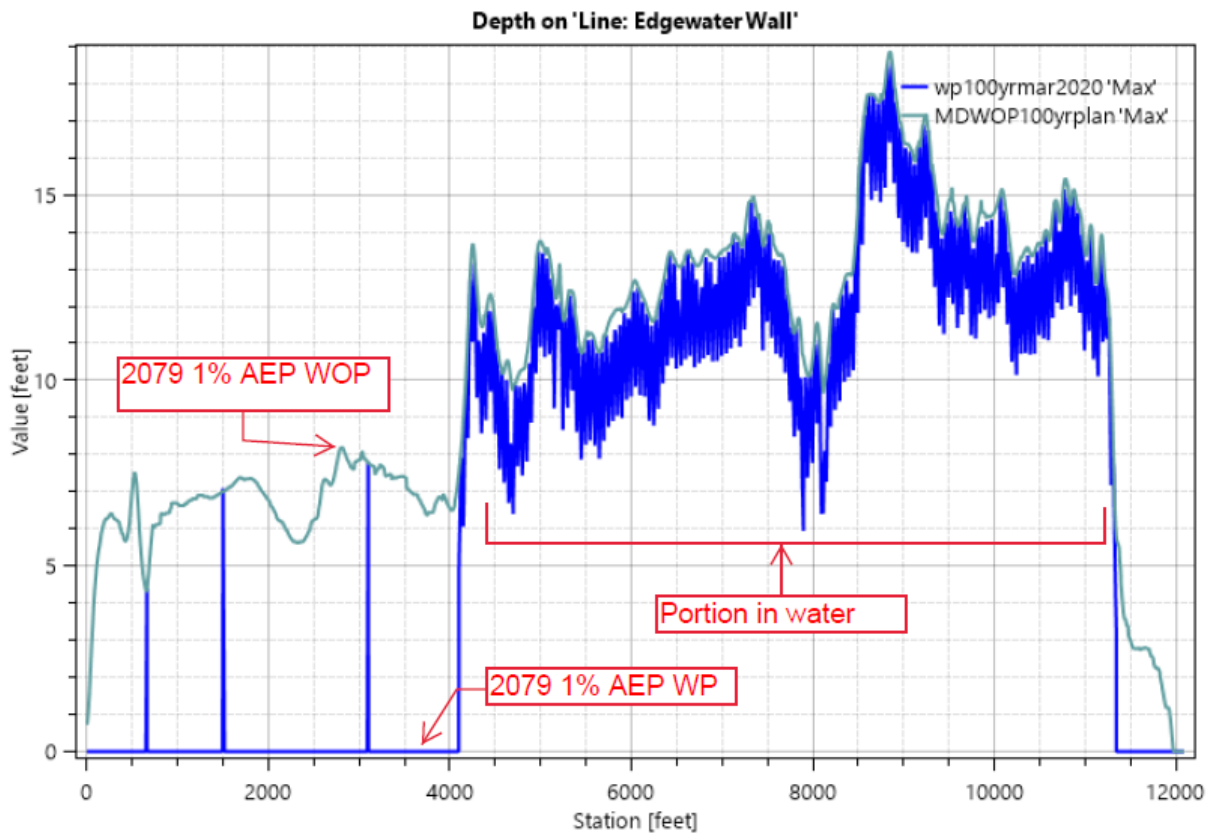


Figure 7.13. Depth profile plot along the Edgewater floodwall in HEC-RAS for the With Project and Without Project at 1% 2079 SWL elevation. Without Precipitation.

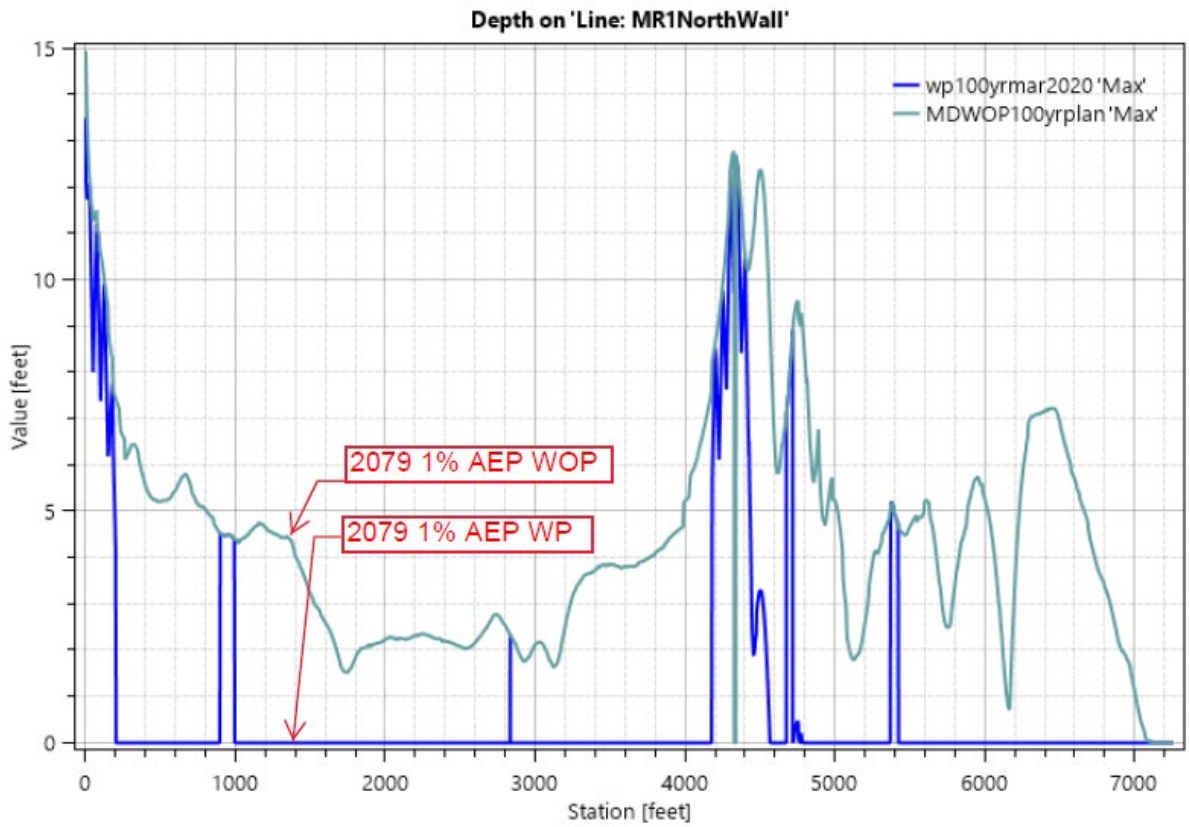


Figure 7.14. Depth profile plot along the Miami River North floodwall in HEC-RAS for the With Project and Without Project at 1% 2079 SWL elevation. Without Precipitation.

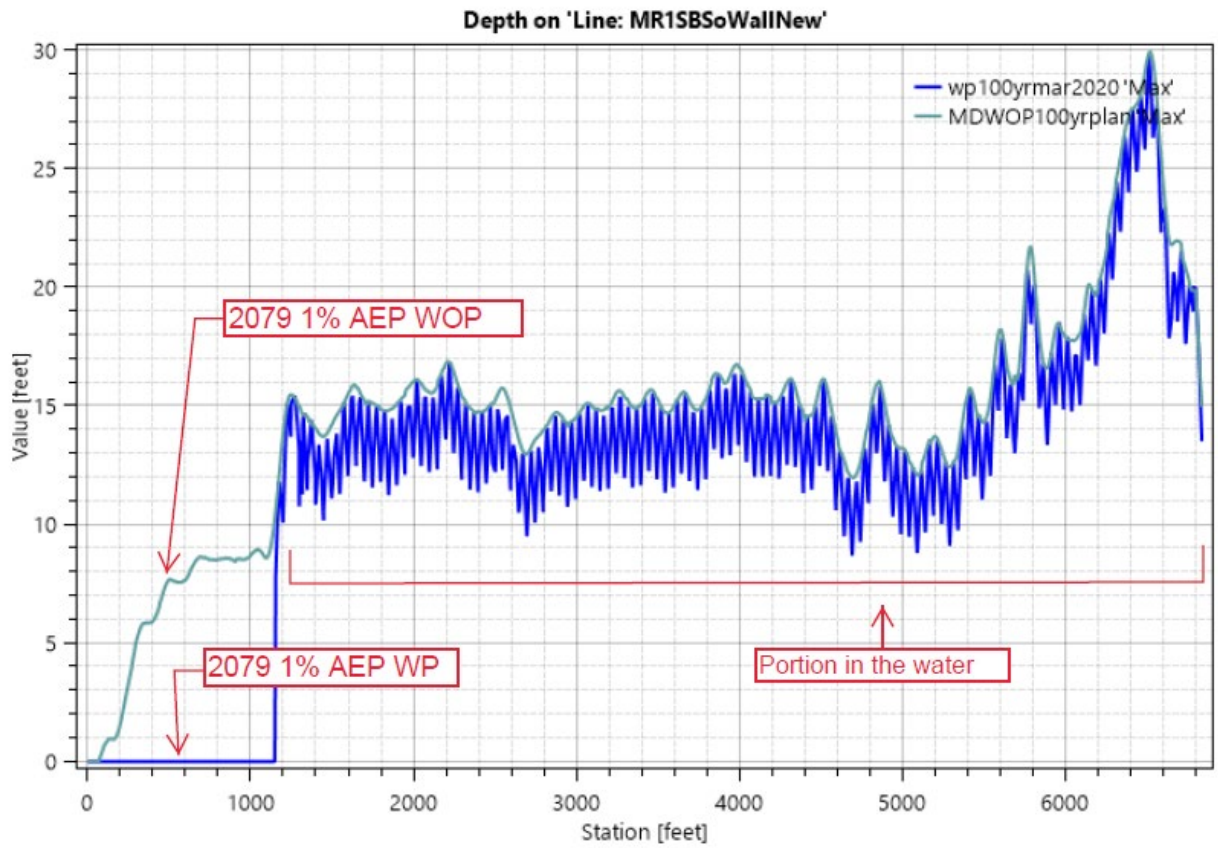


Figure 7.15. Depth profile plot along the Miami River South floodwall in HEC-RAS for the With Project and Without Project at 1% 2079 SWL elevation. Without Precipitation.

7.2Pump Station Locations

The selection of the pump station locations are intended to be near the lowest elevation nearest the floodwall in the watershed. Other possibilities are a drainage channel along the protected side of the wall to channel runoff to the pump station. Miami-Dade County furnished previous analysis reports of the C-4, C-6, C-7, and C-8 watersheds. In these reports, a detailed H&H analysis was done to assess the current pumps and canal capacities for future conditions that include frequency rainfall and various SLR scenarios. In this Miami-Dade County Back Bay CSRM Study, a detailed H&H analysis was not performed before the draft report. A general hydrologic approach to compare rainfall was discussed earlier. A general approach to hydraulic 2D modeling was discussed earlier. The FWOP conditions were considered in the general approaches. Without doing an interior drainage

analysis for this study, the effect the floodwalls and surge barriers have on the watershed is unknown. It is assumed that the minimum sizes needed will at least equal the current recommendations in the Level of Service Assessment reports. **Table 7.2** illustrates what the provided reports from Miami-Dade County have suggested for pump sizes needed.

Table 7.2 Pump Capacities for SFWMD structures

Focus Area	SFWMD structure	Current capacity	Pump 100 year assessment with 3 feet SLR
Miami River (C-4)	S-25B	2000 cfs	3000cfs
Miami River (C-6)	S-26	3470 cfs	not avail
Edgewater	none	none	not avail
Little River (C-7)	S-27	2800 cfs	3300 cfs
Biscayne Canal (C-8)	S-28	3220 cfs	not avail
Arch Creek	G-58	300 cfs	not avail

As a result of the structural solutions discussed, the following paragraphs discuss recommendations for the focus areas. For the Miami River surge barrier, it is possible to have the pump station integrated with the storm surge barrier or have it a short distance away. For the Arch Creek watershed, upgrading existing Stormwater Pump Stations should also be considered. Also in the Arch Creek watershed, the current SFWMD structure G58 is recommended to be upgraded to relieve the interior. In the Miami South wall alignment the existing bulkheads appeared to drain away from the river towards the drain. Upgrading the existing SFWMD control structures should also be considered.

For the areas south of the Miami River watershed, the modeling indicates that there is a possibility of a recommended relief system near the C3 watershed on Coral Gable Water Way near Cartagena Park and on the canal with the SFWMD S22 structure. The SFWMD operates the flood control system to draw down water levels before a hurricane or strong coastal storm approaches. The SFWMD operating rules during storm events need to be facilitated or cannot be impeded. During the PED phase the operation coordination of the

storm surge barriers and pump stations, size of the pump stations, and the best location for the pump station or relief system will be determined during the interior drainage analysis.

7.3Interior Drainage Overview

To account for groundwater for the simplistic approach in this study, saturated conditions and baseflow were included in the hydrologic model assumptions to yield a higher runoff volume for a conservative approach. A more detailed H&H analysis is recommended for the PED phase. As a result the H&H analysis will most likely have smaller watersheds to more accurately account for runoff, groundwater, and the operations of the SFWMD gated structures.

CHAPTER 8 Storm Surge Barrier Closure Analysis

This chapter will discuss how often the storm surge barriers could potentially remain closed during a coastal tidal surge event. This analysis will assume that the duration of the closure will be the same for each proposed area. According to Mooyaart and Jonkman (2017) storm surge barriers are defined as fully or partly movable barriers that can be closed temporarily to limit water levels in the basin behind the barrier, preventing flooding of the area surrounding the inner basin. During normal conditions, the barriers are kept open to all tidal exchanges and navigation. Eighteen storm surge barriers have been built worldwide, with interest to construct more along areas vulnerable to tidal flooding from storm surge. This analysis will start by choosing a historical storm of record to base the assumption of anticipated days of closure on.

The National Weather Service (NWS) gauge – Atlantic Ocean Tide Gauge at Virginia Key has action stages associated with it. The datum of the NWS gauge is Mean Higher High Water (MHHW). At 1.0 feet MHHW (0.79 ft. NAVD88) is the Action stage up to 2.5 feet MHHW (2.29 ft. NAVD88) for a Major stage. The datum conversion at the NOAA Virginia Key gauge is 0.0 feet MHHW = -0.21 feet NAVD88.

According to the NWS Atlantic Ocean Tide gauge, Miami-Dade County starts to see damages occur when the SWL elevations reach an elevation of 1.49 feet NAVD88 (the moderate action stage). Examining the report data from NOAA-Virginia Key gauge, **Table 8.1** shows where the water surface elevations reached above 2.0 feet NAVD88. Some of the events on **Table 8.1** are from coastal storms. Storm surge is associated with all cyclonic storms (mainly hurricanes, tropical storms, and nor'easters that are cyclonic in nature).

Table 8.1. Historical Data from NOAA- Virginia Key Gauge.

Highest and Lowest Values

Station:	8723214	Begin Date:	19940101
Name:	Virginia Key, Biscayne Bay, FL	End Date:	20200201
Product:	High/Low	Units:	Feet
Datum:	NAVD	Quality:	Verified

Rank	Highest	Highest Date	Zone	Lowest	Lowest Date	Zone
1	3.79	20170910 17:00	GMT	-3.31	19940329 21:42	GMT
2	2.77	20051024 12:30	GMT	-3.08	19960217 06:12	GMT
3	2.24	20171005 13:06	GMT	-2.94	19940328 21:00	GMT
4	2.19	20191001 15:54	GMT	-2.93	19980101 09:42	GMT
5	2.18	20170910 04:24	GMT	-2.91	20010110 08:06	GMT
6	2.15	20050920 16:00	GMT	-2.91	20090110 06:42	GMT
7	2.13	19941115 11:12	GMT	-2.91	20090111 07:36	GMT
8	2.12	20121028 12:48	GMT	-2.90	20030119 08:24	GMT
9	2.09	19991015 19:42	GMT	-2.89	20041215 10:12	GMT
10	2.08	20190930 15:00	GMT	-2.89	19960308 22:12	GMT

After observing the existing data, if gates were in place during each of the storm events shown in **Table 8.1**, the gates would be closed the longest during a Hurricane Irma type storm event, due to the water surface elevations remaining above 2.29 feet NAVD88. Therefore, Hurricane Irma in September 2017 will be the storm analyzed for the basis of this storm surge barrier analysis. **Figure 8.1** is a plot of the water surface elevation during the Hurricane Irma storm event. During Hurricane Irma, at the Virginia Key tide gauge, the water level was above 2.29 feet NAVD88 for about 7-8 hours.

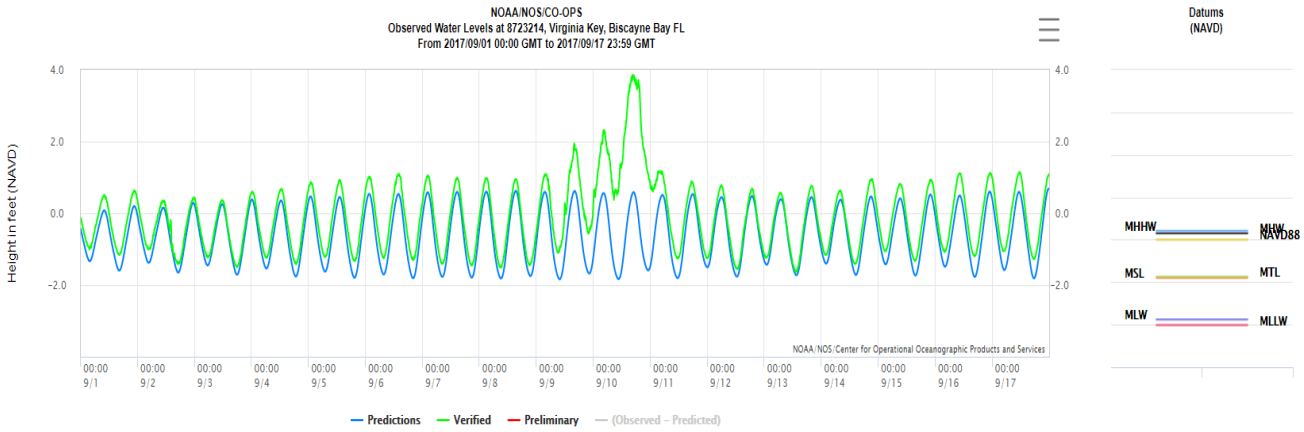


Figure 8.1. Plot of Hurricane Irma Water levels (NAVD88) measured at Virginia Key.

Since the Stillwater elevations with storm surge for the tentatively selected plan changes, due to the effects of relative sea level rise over the project economic period of analysis, the changes in the sea level will be added to the data that make up **Figure 8.1**. **Figure 8.2** displays the plot of **Figure 8.1** and shows how the water levels during Hurricane Irma would be observed with the appropriate change in relative sea level rise at the end of the project economic period of analysis (year 2079).

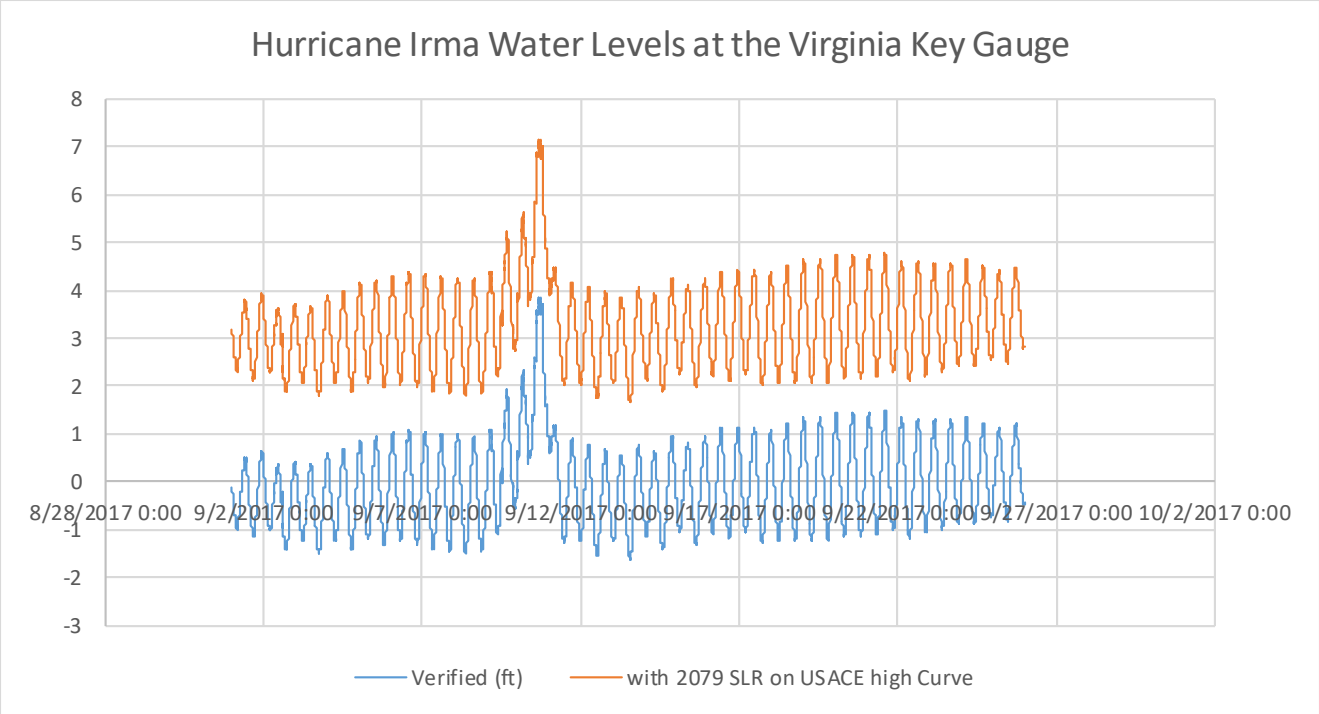


Figure 8.2. Plot of Hurricane Irma Water levels (NAVD88) measured at Virginia Key gauge and w/RSLR.

After observing the existing data and adding the appropriate increase in relative sea level rise, **Table 8.2** displays approximately how many days the storm surge barriers would be closed if a storm such as Hurricane Irma were to occur for the end of the project economic period of analysis (year 2079). The intent is to ensure that barriers are closed before damages would occur; therefore, the surge barriers would need to be closed at a low tide elevation. It was assumed (resulting in **Table 8.2**) that the barriers would be closed at low tide approximately 12 hours before the 2.29 foot elevation (in which damages could occur) is expected to rise. For the with 2079 SLR condition, the time that the elevation was

continuously above 2.29 ft. NAVD88 was approximately 32 hours (1.33 days) plus the assumed 12 hours before yielded 44 hours (1.83 days). Rounding up to whole days brought the closure to two days for a conservative approach. It was also assumed two days will be the same for all surge barriers in this project.

Table 8.2. Approximately Closure Time (in days).

Year	Closure Time in days (approx.)
2017 (Irma)	1 day
2079	2 days

The results of this brief analysis show that as the climate continues to change and sea level rise continues to increase over time, the longer the surge barriers will have to be closed for a given storm event over time, to prevent damages from occurring.

How often and frequent the barriers would need to be closed in a given year is difficult to predict, since one will need to forecast future weather events. Future storm events and their intensity are unknown, which is why the water surface elevations are the only thing analyzed in frequencies and probabilities as they related to storm events. To positively predict and interpret the occurrence of future storm events and their intensities Lane (2008) explains that three things have to be understood: event magnitude, AEP, and spatial scale. Climate Change scientist believe that storm events are occurring more frequently and becoming more intense. In order to accurately assess any changes in the frequency of these events, long term monitoring and data collection is required. Extreme events are still rare compare to the history of recorded data and observing enough of these events to form any statistically viable conclusions (or frequency curves) is going to take many years (Climatica 2018). For example, measurements of storm intensity are very accurate from the 1970s to the present day due to the increasing use of satellites and monitoring networks such as NOAA. Prior to this, however, our records of storm occurrence and frequency are far less accurate due to incomplete written records, and the fact that many storms occur over the oceans, where there are very few human populations who might monitor and record such events and in addition to that our global record are also highly spatially variable as data from less inhabited or developing regions is sparse. Only once the quality and length of our records improve will

we be able to more fully understand, and better predict, extreme event frequency for storms (Climatica 2018). The frequency of storm surge barriers can only be approached, currently, by observing current frequency curves or water surface elevations. The frequencies can be observed in Miami-Dade County. It can be said the barriers would close for a high frequency coastal storm event in which the forecasted Stillwater elevation is predicted to be above 2.29 feet NAVD88 (the NWS major action stage) or at a less frequent storm event, but only when the storm surge event of 2.29 feet NAVD88 or higher is expected. For example, the tentatively selected plan for this study proposes a project alignment, which protects against 1% storm event. This means that we have 1% chance in any given year of a storm occurring that would produce Stillwater levels (which include storm surge) for a 1% storm. **Figures 2.6 to 2.10** of this sub-appendices displays frequency curves of the Stillwater level for all four study areas. These curves display the Stillwater, including storm surge, for the year 2018 (the start of the study).

Chapters 2 and 3 of this report discuss the water levels for each area at the start of the study (year 2018) and to the end of the project economic period of analysis, year 2079. There are several existing storm surge barriers in the country and around the world, with historical records that show frequency of closures varied throughout the life of the projects. The proposed structural design of the sector gate for storm surge barrier for the Miami-Dade Back Bay CSRM was similar that of the existing surge barrier constructed by the U.S. Army Corps of Engineers in New Bedford, Massachusetts (**Figure 8.3**).

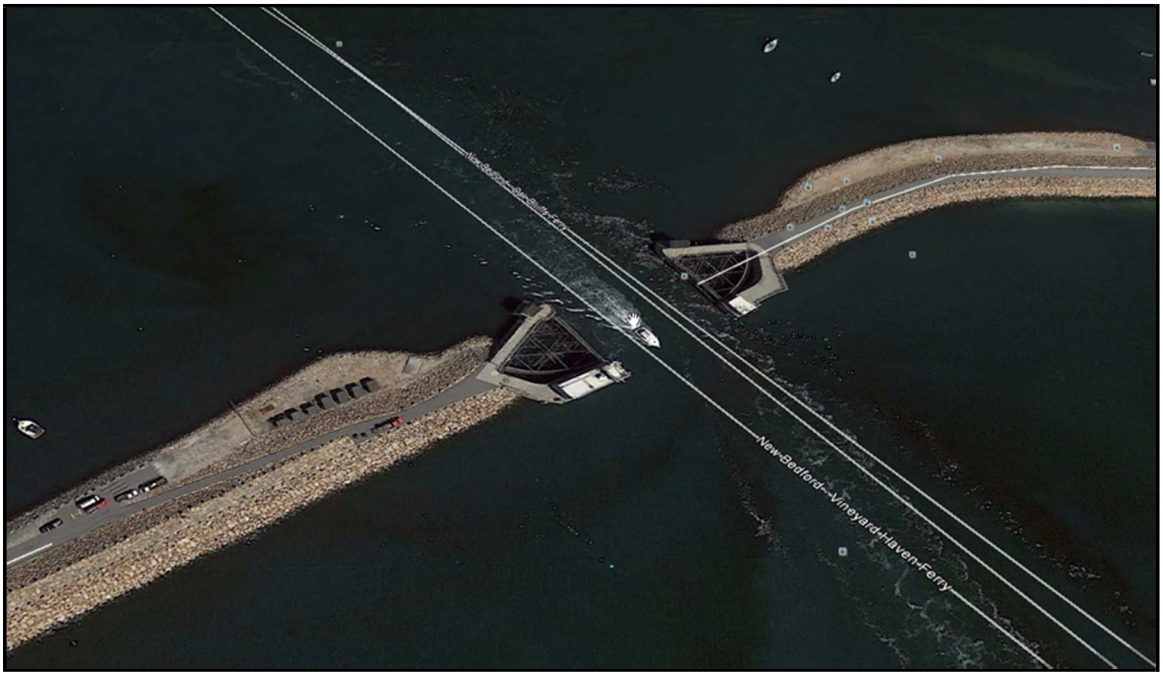


Figure 8.3. Bird's Eye View of the New Bedford – Fairhaven Barrier (Google Earth Image).

This barrier consists of a 9,186 foot dam with a crest level of more than 20 feet above mean sea level, a navigation opening of around 151 feet wide is protected by two sector gates. The sector gates have a height of approximately 60 feet and are housed in side chambers in the abutments and are rolled using steel wheels on a concrete sill. The gates take approximately 12 minutes to close. This project can handle the storm surge up to 20 feet. Another storm surge barrier in New England is the Stamford Hurricane Protection Barrier located in Stamford, Connecticut (**Figure 8.4**).

Figure 8.4. Aerial View of Stamford Hurricane Protection Barrier (Courtesy of USACE – New England District).

Construction for this barrier was completed in January 1969. This barrier, which has an elevation of 17 feet, is composed of a 1,350-foot-long concrete wall; 2,950 feet of earth fill dike with stone slope protection and a pumping station. The third portion provides protection at Westcott Cove. This barrier is a 4,400-foot earth filled dike with stone slope protection having a maximum elevation of 19 feet. It also has two pumping stations. A third barrier is the Fox Point Hurricane Barrier, located in Providence, Rhode Island. Construction of this barrier was completed in 1966 (**Figure 8.5**). The Fox Point barrier itself is a 700-foot-long concrete structure, 25 feet high, and Two 10 to 15-foot-high earth fill dikes with stone slope protection, flank each side of the barrier. The eastern dike is 780 feet long and the western dike is 1,400 feet long. It also has a pumping station and cooling water canal as integral parts of the project. During a tidal/flood situation, the pumping station's five large pumps can discharge the flood waters of the Providence River through the barrier into the bay. Two gated openings in the pumping station, each 10 feet high and 15 feet wide, admit water into the cooling water canal used by the Narragansett Electric Company, located immediately

behind the barrier. **Table 8.3** shows a record of how many times each of these three barriers were closed. The record shows that it varies from year to year.

Figure 8.5. Image of Fox Point Barrier (Image courtesy of Wikipedia).

Table 8.3. New England Storm Surge Barriers Closure record.

Fiscal Year	Stamford	New Bedford	Fox Point *		Fiscal Year	Stamford	New Bedford	Fox Point *
1966	--	4	N/A		1993	12	20	N/A
1967	--	2	N/A		1994	3	26	N/A
1968	--	2	N/A		1995	1	17	N/A
1969	7	3	N/A		1996	9	40	N/A
1970	10	5	N/A		1997	10	31	N/A
1971	15	7	N/A		1998	19	25	N/A
1972	25	18	N/A		1999	8	17	N/A
1973	24	12	N/A		2000	10	9	N/A
1974	13	6	N/A		2001	14	11	N/A
1975	16	5	N/A		2002	7	6	N/A
1976	7	7	N/A		2003	11	12	N/A
1977	7	6	N/A		2004	3	10	N/A
1978	22	9	N/A		2005	14	25	N/A
1979	16	11	N/A		2006	14	19	N/A
1980	12	11	N/A		2007	18	16	N/A
1981	5	1	N/A		2008	10	15	N/A
1982	8	4	N/A		2009	12	14	N/A
1983	9	5	N/A		2010	22	38	15
1984	14	7	N/A		2011	16	29	12
1985	7	4	N/A		2012	17	16	12
1986	9	3	N/A		2013	26	29	10
1987	16	4	N/A		2014	13	28	2
1988	5	0	N/A		2015	9	11	4
1989	7	4	N/A		2016	18	32	12
1990	2	4	N/A		**2017**	**16**	**40**	**10**
1991	2	12	N/A					
1992	4	11	N/A		**TOTAL:**	**574**	**703**	**77**

* Corps took over Fox Point Barrier in February 2010

Another barrier with an extensive period of record is the Thames Surge Barrier in London, England (**Figure 8.4**).

Figure 8.4. Thames Barrier (image courtesy of Wikipedia).

The Thames Barrier protects the city of London against floods. It consists of ten openings with spans ranging from approximately 98 feet to 200 feet. Six rotary segment gates are applied in the larger openings to allow navigation. The other four openings consist of normal segment gates.

The length of the barrier is approximately 1,739 feet and was constructed between 1974 and 1982. Table 8.3, below, shows the number of time the Thames Surge Barrier needed to be closed during past storm surge events. It should be noted that the purpose discussing the different surge barriers is to explain that no record from any storm barrier shows a consistent pattern or that frequencies of propose gate closures cannot easily be predicted. In no way, shape or form should the designs and operations of these barriers be a consideration of how the proposed Miami-Dade surge barrier will perform. For coastal engineering projects, *solutions are generally site specific.*

Table 8.3. Thames Surge Barrier Closure Frequency.

Season (Sep–Apr)	Closures per season			Season (Sep–Apr)	Closures per season		
	From tidal flooding	From fluvial flooding	Total[8]		From tidal flooding	From fluvial flooding	Total[8]
1982–83	1	0	1	2000–01	16	8	24
1983–84	0	0	0	2001–02	3	1	4
1984–85	0	0	0	2002–03	8	12	20
1985–86	0	1	1	2003–04	1	0	1
1986–87	1	0	1	2004–05	4	0	4
1987–88	0	0	0	2005–06	3	0	3
1988–89	1	0	1	2006–07	8	0	8
1989–90	1	3	4	2007–08	6	0	6
1990–91	2	0	2	2008–09	1	4	5
1991–92	0	0	0	2009–10	2	3	5
1992–93	4	0	4	2010–11	0	0	0
1993–94	3	4	7	2011–12	0	0	0
1994–95	2	2	4	2012–13	0	5	5
1995–96	4	0	4	2013–14	9	41	50
1996–97	1	0	1	2014–15	1	0	1
1997–98	1	0	1	2015–16	1	0	1
1998–99	2	0	2	2016–17	2	0	2
1999–00	3	3	6	2017–18	3	0	3

As stated earlier in this chapter Stillwater elevation frequencies or probabilities can be determined for a particular project. Therefore, the storm surge barriers could close at approximately at 2.29 feet NAVD88 Stillwater elevation (the NWS major action stage) or less frequent storm event. We can expect storms of lower frequencies to occur. Based on historical storms we can anticipate a storm having storm surge of 2.29 feet NAVD88 or greater to occur 0 to 10 times a year. Also, meaning that we can anticipate the predicted days closed to occur approximately 4 to 30 days (+/- 1 day) in one year. How that number may vary over time and the variance of that number at this point is unknown. Water Quality Modeling is recommended to see what events the storm surge barriers could have on aquatic life. The discussion of Water Quality modeling can be read in the Environmental Appendix.

CHAPTER 9 Adjustments during Preconstruction Engineering and Design (PED)

During PED, the HH&C analysis will be refined and also result in refinement of the design. This chapter will discuss what analysis will be performed during PED.

9.1 Hydrodynamic Modeling of Structural Measures

During the PED phase of this project, additional hydromodeling will be used to simulate conditions for a range of conditions (waters levels, winds, etc.). The South Atlantic Coast Study (SACS) will likely be completed prior to the PED phase and the results from this study can be compared to those from the SFLSSS referenced in this report. The hydromodeling process will be applied to each alternative plan to determine the behavior of the surge and waves; the interaction between structural measures, coastal features, incoming surge and waves during a storm event, and the likelihood of flooding that could occur inside of the project alignment from overtopping and rainfall during a storm event. When the initial hydromodeling begins for the measures, a step-wise procedure will be used.

The steps are as followed:

- Hydromodeling Step 1: Surge Levels and Wave Characteristics
  During the PED phase, the ADCIRC grid that was used for the SACCS modeling, should be used to model the recommended plan structures. The proposed structures of the project plan will be added to the ADCIRC grid in the with-project runs. These runs will be compared to without-project runs. The with-and without-project runs will include storms from the SACCS storm suite with different characteristics (direction, waves, velocity, water levels, rainfall intensity, etc.) that produce storm surges that will range from the design water level to elevations that exceed the design water level. The comparison between the with- and without-project modeling results may result in modifications to the project alignment. At this time we do not know how many simulations will be executed. The project report will include further discussion on how the project structures will be added to the ADCIRC model, how and which storms will

be selected for modeling, and if tropical or extra-tropical storms should be modeled. High tides are currently included in the design Stillwater estimates and they will continue to be so in the PED modeling.

- Hydromodeling Step 2: Frequency Analysis
  This step will be performed by the SACCS modeling efforts and as a result the frequency analysis resulted in stage frequencies for different areas around the Miami-Dade County. The analysis includes surge, waves, and tides.

- Hydromodeling Step 3: Overtopping Analysis & Volumes
  Wave overtopping analysis was performed by using the computer aided software ACES. SFLSSS Stillwater levels were computed for four AEPs (5% flood, 2% flood, 1% flood, and 0.5% flood frequencies). This step was performed by using the computer, however, to get a holistic picture of during PED, hydromodeling will be performed to confirm and check the values from the ACES software and compare those values using results from the SACCS.

- Hydromodeling Step 4: Determination of Interior Flooding
  The existing conditions would have to be modeled for each of the areas that have structures proposed. The existing condition model will need to be calibrated to historical storms. Once calibrated, the model can be stressed to model the future conditions for with and without the project. The difference between the last two conditions would determine the relief capacity of the interior area. Once determined, then sizing of relief option (pump station, gravity outlets, etc.) can be determined.

The USACE policy of using the Engineer Manual for Hydrologic Analysis of Interior Areas (EM 1110-2-1413), dated 24 August 2018, would need to apply.

9.2 ADDITIONAL CONSIDERATIONS

Hydraulic Modeling: Determination of the Extents of the Hydromodeling

The Hydraulic segment (USACE will utilize HEC-RAS) of the modeling will be highly important to understand the extents of the inundation, the depth of the inundation, and the effect of the structural solutions. Recommend a high resolution DEM that includes the bathymetry of the Biscayne Bay and the interior canals. It is recommended to have surveyed bathymetry to include recent harbor deepening and/or dredging projects to yield proper depths of inundation.

It is likely the hydraulic modeling will reveal more hydraulic connections from the western side of Miami-Dade County which could reveal more complex solutions for the 1% AEP SWL elevation that was considered in this study.

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