PRELIMINARY BRIDGE HYDRAULICS REPORT

TOWN OF BAY HARBOR ISLANDS

BROAD CAUSEWAY BRIDGE REPLACEMENT
PROJECT DEVELOPMENT & ENVIRONMENT STUDY

Prepared for:

Town of Bay Harbor Islands, Florida

March 7, 2024

Financial Project Identification Number:	452428-1-21-01
Federal Project Number:	N/A
FDOT Efficient Transportation Decision Making (ETDM) Number:	14520
Town of Bay Harbor Islands Project Number:	BC-160

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March 7, 2024

The environmental review, consultation, and other actions required by applicable Federal environmental laws for this project are being or have been carried out by Florida Department of Transportation (FDOT) pursuant to 23 U.S.C. §327 and a Memorandum of Understanding dated May 26, 2022, and executed by Federal Highway Administration (FHWA) and FDOT.

Prepared for:
Town of Bay Harbor Islands

Prepared by:
Atkins

PRELIMINARY BRIDGE HYDRAULICS REPORT

Florida Department of Transportation

District 6

In cooperation with the Town of Bay Harbor Islands

Financial Management Number: 452428-1-21-01

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Broad Causeway Bridge Replacement Project Development and Environment (PD&E) Study

Broad Causeway Bridge from Broad Causeway Island to East of West Broadview Drive

Miami-Dade County, Florida

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PROFESSIONAL ENGINEER CERTIFICATE

I hereby certify that I am a registered professional engineer in the State of Florida practicing with AtkinsRéalis, and that I have supervised the preparation of, and approved the evaluation, findings, opinions, conclusions, and technical advice reported in:

REPORT:	Preliminary Bridge Hydraulics Report
PROJECT:	Broad Causeway Bridge Replacement PD&E Study
LOCATION:	Miami-Dade County, Florida
FINANCIAL MANAGEMENT NO.:	452428-1-21-01
FEDERAL PROJECT NO.:	N/A
FDOT ETDM NO.:	14520

This Preliminary Bridge Hydraulics Report (BHR) contains engineering information that fulfills the purpose and need for the Broad Causeway Bridge Replacement PD&E Study from Broad Causeway Island to East of West Broadview Drive in Miami-Dade County, Florida. I acknowledge that the procedures and references used to develop the results contained in this report are standard to the professional practice of transportation engineering as applied through professional judgment and experience.

I hereby certify that I am a registered professional engineer in the State of Florida practicing with AtkinsRéalis, and that I have prepared or approved the evaluation, findings, opinions, conclusions, or technical advice for this project.

This item has been digitally signed and sealed by *Michael Salisbury, P.E.* on the date adjacent to the seal.

Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.

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EXECUTIVE SUMMARY

The overall purpose of this project involves the potential replacement of the Broad Causeway Bridge connecting the Town of Bay Harbor Islands (Town) with the City of North Miami, within Miami-Dade County. The bridge is part of Broad Causeway, a roadway classified as "Urban Minor Arterial". This arterial also begins in Bal Harbour/Surfside and connects those commuters to the mainland. The existing bridge consists of four lanes, undivided (two in each direction), the four travel lanes are 10 ft. wide, without a raised median. The outside travel lanes also include shared-use markings to accommodate bicycles. In addition, a raised sidewalk is present on each side of the bridge, with a width that varies from 22 to 36 inches. The existing bridge, constructed in 1951, has been determined to be functionally obsolete, and contains fracture critical components based on a Bridge Inspection Report prepared in January 2023 and determined to be structurally deficient based on a Bridge Inspection Report prepared in January 2024 by FDOT. As a result of the 2024 inspection, temporary emergency repairs will be completed. One lane of the bridge is closed until repairs are complete. It is expected that major costly repairs will be needed more frequently as the bridge ages to prevent closure or severe damages. Because of the structure type, the number of structural deficiencies, and high maintenance costs, the Town is considering replacement of the bridge.

The project site is located between the mainland and Bay Harbor Islands in Miami-Dade County, FL and the region is prone to tropical storm activity. There are two inlets providing conveyance with the Atlantic Ocean: Haulover Inlet to the north and Government Cut to the south. Due to the connection to the Atlantic Ocean via the two inlets, scour conditions resulting from storm surge are expected at the site. Note that there aren't any major rivers nearby, so scour due to river discharge is not expected to be a concern.

In order to calculate scour, information on the currents passing by the bridge needs to be understood. Since there are no *in situ* water level and current measurements at the bridge site, numerical modeling was performed to determine reasonable design conditions that are unique to this particular site. In light of this, a Delft3D hydrodynamic and wave model was developed and implemented to simulate waves and storm surge as they propagate from the Atlantic Ocean to the Broad Causeway bridge site, including the 50-year, 100-year and 500-year storm events.

The wind fields were scaled to represent various return period storm events (50-year, 100-year, and 500-year). Using the model results from these storm events, the contraction scour and local scour were computed at the proposed bridge piers. The summary table below provides the total predicted scour for the 50-year, 100-year and 500-year events. This information can be used to help guide the planning process for the proposed Broad Causeway bridge project.

Table ES-1 Summary of calculated scour depths at the Broad Causeway Bridge location.

Type of Scour	50-year Scour Depth (ft)	100-year Scour Depth (ft)	500-yr Scour Depth (ft)
Long-term	0	0	0
Contraction	1.6	1.0	0.4
Local	6.7	6.2	6.2
Total Scour	8.3	7.2	6.6
Scour Elevation*	-16.6	-15.5	-14.9

**Based on existing channel elevation of -8.3 ft (NAVD88).*

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ACRONYMS AND ABBREVIATIONS

A	**ADA**	Americans with Disabilities
C	**CUDEM**	Continuously Updated Digital Elevation Model
D	**D_{50}**	Median Grain Size
F	**FDOT**	Florida Department of Transportation
	FEMA	Federal Emergency Management Agency
	FHWA	Federal Highway Administration
	FIRM	Flood Insurance Rate Maps
	FIS	Flood Insurance Studies
I	**ICWW**	Intracoastal Waterways
M	**MHW**	Mean High Water
	MHHW	Mean Higher-High Water
	MLW	Mean Low Water
	MLLW	Mean Lower-Low Water
	MSL	Mean Sea Level
N	**NGS**	National Geodetic Survey
	NMI	Nautical Miles
	NOAA	National Oceanic and Atmospheric Administration
P	**PD&E**	Project Development and Environment
R	**ROW**	Right-of-way
	RMS	Root-Mean-Square
U	**USCG**	United States Coast Guard

1.0 PROJECT SUMMARY

1.1 Project Description

The project involves the potential replacement of the Broad Causeway Bridge connecting the Town of Bay Harbor Islands (Town) with the City of North Miami, within Miami-Dade County. The bridge is part of the Broad Causeway, a roadway classified as "Urban Minor Arterial". This arterial also begins in Bal Harbour/Surfside and connects those commuters to the mainland. The specific limits of the project extend from the Broad Causeway Island (25°53'19.41"N, 80° 8'54.52"W) on the west side and (25°53'11.30"N, 80° 8'18.93"W) to east of West Broadview Drive. The improvements include the bridge approaches and Broad Causeway Island circulation. The Florida Department of Transportation (FDOT) Bridge Identification (ID) Number (No.) is 875101. A graphic depicting the location of the bridge is provided as **Figure 1-1**. The project is approximately 0.77 mile in length.

The existing bridge consists of four lanes, undivided (two in each direction), the four travel lanes are 10 ft. wide, without a raised median. The outside travel lanes also include shared-use markings to accommodate bicycles. In addition, pedestrians use a raised maintenance area on each side of the bridge, with a width that varies from 22 to 36 inches. There are no guardrails separating the sidewalk from the travel lane. Crossing over the Intracoastal Waterway (ICWW), the bridge has a horizontal clearance of 79.7 ft., a maximum vertical clearance of 18.0 ft. at Mean Low Water (MLW) and a minimum vertical clearance of 15.7 ft. at Mean High Water (MHW) at the Bascule crossing. The ICWW at the bridge crossings is deemed a navigable waterway by the United States Coast Guard (USCG). The bridge bascule is required by the USCG to open twice per hour on the quarter and three-quarter hour but only opens if vessels are waiting.

The existing bridge, constructed in 1951, has been determined to be functionally obsolete with fracture critical components based on a Bridge Inspection Report prepared in January 2023 and determined to be structurally deficient based on a Bridge Inspection Report prepared in January 2024 by FDOT. In 2017, major structural repairs were performed to the bridge at a construction cost of approximately $17 million. As a result of a 2020 inspection carried out by FDOT, a design to address additional repairs has been completed and it has been determined that the cost to perform these repairs will amount to $3.0 million. As a result of the 2024 inspection, temporary emergency repairs will be completed. One lane of the bridge is closed until repairs are complete. It is expected that major costly repairs will be needed more frequently as the bridge ages to prevent closure or severe damages. Because of the structure type, the number of structural deficiencies, and high maintenance costs, the Town is considering replacement of the bridge.

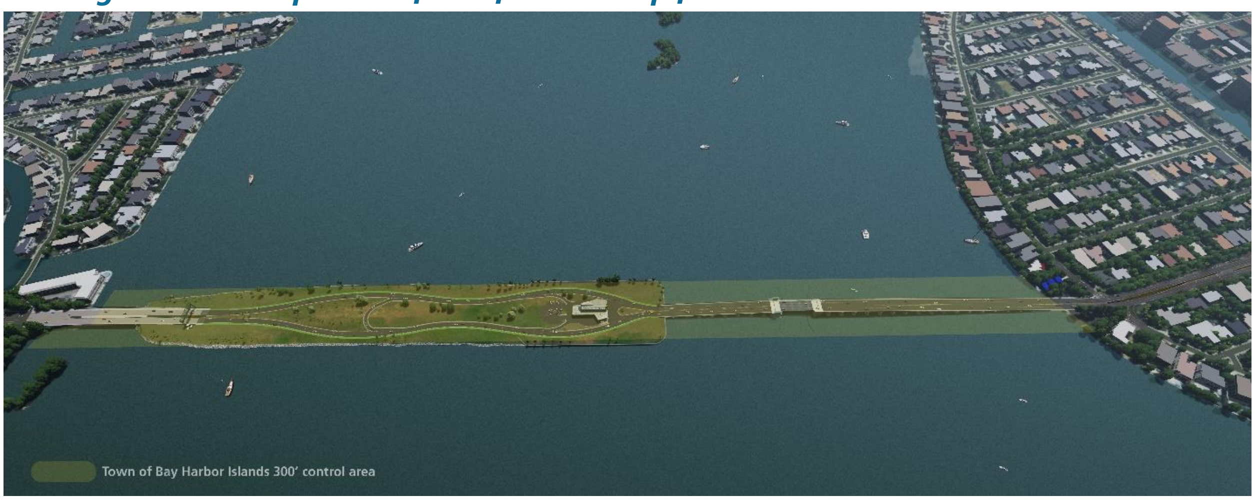
This Project Development and Environment (PD&E) Study has been conducted to address the structural and functional deficiencies of the existing Broad Causeway Bridge and to evaluate and compare the feasibility of continued rehabilitation and repair versus replacement of the bridge.

Figure 1-1 Project Location Map

Bridge concepts will include provisions for new pedestrian and bicycle accommodations to comply with Americans with Disabilities Act (ADA) requirements and guardrails for the safety of pedestrians.

Existing right-of way (ROW), owned by the Town, is anticipated to accommodate the replacement bridge and approaches. Included in the Town Charter by the 1953 Senate Bill No. 865, the State of Florida surrendered and granted to the Town any claim or control over all tidewaters and other lands, and all bayous and bay bottoms, beaches, waters, waterways and water bottoms, and all riparian rights within and adjacent to the Town limits for municipal purposes only, a strip of 300 ft. wide from Kane Concourse, westwardly across Biscayne Bay to approximately 123rd Street in the City of North Miami. This 300-ft. wide strip is shown in **Figure 1-2** as a bright yellow highlight. Therefore, the replacement bridge will be built within the 300 ft. strip over Biscayne Bay under claim or control by the Town.

Figure 1-2 Depiction of 300-ft. wide strip from Kane Concourse to North Miami



1.2 Purpose and Need

The purpose of this project is to address the structural and functional deficiencies of the existing Broad Causeway Bridge. The need for the project is to improve bridge deficiencies because the 73-year-old bridge is structurally deficient, functionally obsolete, and contains fracture critical components; improve safety since there have been several vehicular crashes in the project corridor, many involving bicycles and pedestrians that resulted in injuries; improve flow of traffic along the project corridor which has high traffic volumes and frequent bridge openings; and to maintain emergency evacuation.

A key component in the design of the new bridge is understanding the magnitude of scour at each of the bridge piers and abutments for various conditions, so that structural engineers can properly design the bridge foundation. In order to calculate scour, information on the currents passing by the bridge needs to be understood. Since there are no *in situ* water level and current measurements at the bridge site, numerical modeling must be used to determine reasonable design conditions that are unique to this particular site. In light of this, a Delft3D hydrodynamic and wave model was developed and implemented to simulate waves and storm surge as they propagate from the Atlantic Ocean to the Broad Causeway bridge site for a number of extreme return period storm events. These model results were then used to estimate scour around the proposed bridge piers.

2.0 SITE CHARACTERIZACTION

2.1 General Description

Figure 1-1 shows the Broad Causeway bridge between the mainland and Bay Harbor Islands in Miami-Dade County, FL. The two closest inlets with conveyance between the Atlantic Ocean are to the north at Haulover Inlet and to the south at Government Cut. Due to the connection to the Atlantic Ocean via the two inlets, scour conditions resulting from storm surge are expected at the site. Note that there aren't any major rivers nearby, so scour due to river discharge is not expected to be a concern. At NOAA Station 8723214 in Virginia Key, Biscayne Bay (see **Figure 2-1**), the mean tidal range is about 2.2 ft with mean sea level (MSL) equal to -0.89 ft NAVD88 (NOAA, 2024a). Tides are semi-diurnal in frequency.

2.2 Water Levels

Table 2-1 presents the tidal datums at the Broad Causeway bridge based on NOAA's VDatum tool. In addition, NOAA Station 8723214 in Virginia Key, Biscayne Bay, FL is the nearest tide gauge with long-term measured water level. This station is located to the south of the Broad Causeway bridge and provides a good proxy for site-specific water level information. The tide station was established in 1994, so almost 30 years of data are available for analysis. The minimum water level recorded is -3.27 ft NAVD88 (March 29th, 1994) and the maximum historical water level recorded at this station is 3.83 ft NAVD88 (September 10th, 2017), corresponding to Hurricane Irma which caused severe damage in many parts of the state.

Note that site-specific water level and current information at Broad Causeway bridge was not collected as part of this effort. **It is strongly recommended that this site-specific data be collected during the next phase of the project to support model validation.**

Table 2-1 Tidal datums at Broad Causeway bridge based on NOAA's VDatum tool.

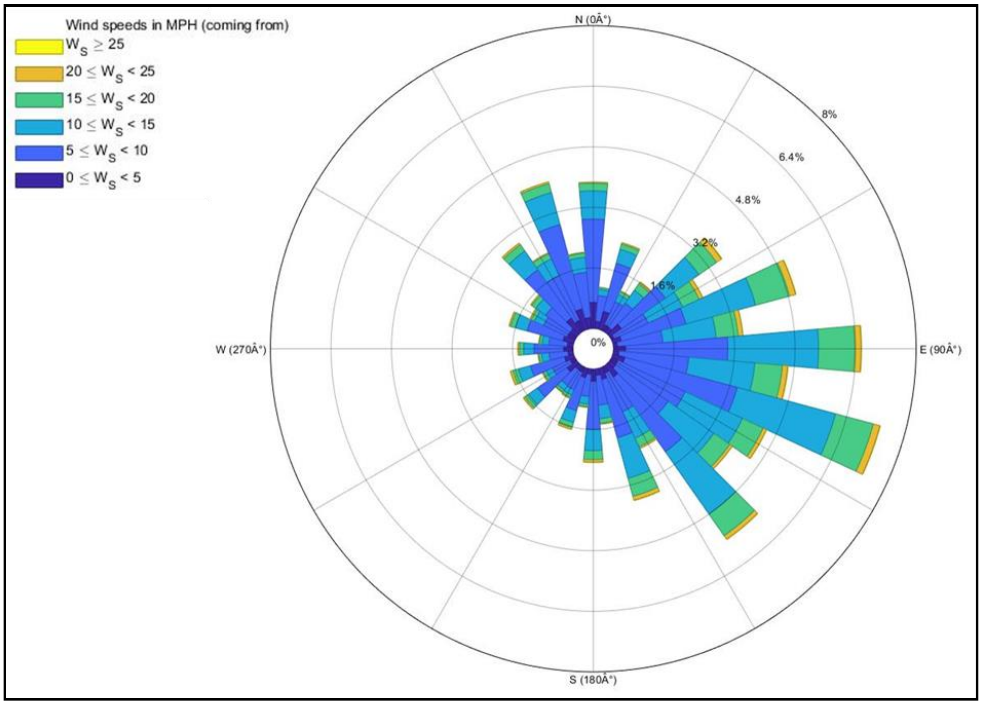
Tidal Datum	*Elevation (ft NAVD88)*
Mean Higher-High Water (MHHW)	*0.26*
Mean High Water (MHW)	*0.19*
Mean Sea Level (MSL)	*-0.89*
Mean Low Water (MLW)	*-1.94*
Mean Lower-Low Water (MLLW)	*-2.07*

Figure 2-1 Map showing the location of Broad Causeway bridge, NOAA tide gauge and meteorological station.

2.3 Winds

Typical wind conditions at the site are obtained from a nearby weather station at the Miami International Airport (https://www.ncei.noaa.gov/cdo-web/) which is approximately 10 miles from the project site (**Figure 2-1**). While this station isn't located right at the Broad Causeway bridge, it is close enough to represent a reasonable proxy for site-specific winds. **Figure 2-2** shows the wind rose from 1950- 2010 with the distribution of wind magnitude and direction which appears to be seasonally influenced. The Broad Causeway bridge location is relatively sheltered compared to the open coast and nearby bays, so local wind-generated waves aren't a concern under normal conditions.

Figure 2-2 Annual wind direction distribution (in percentage) at the Miami International Airport wind gauge.



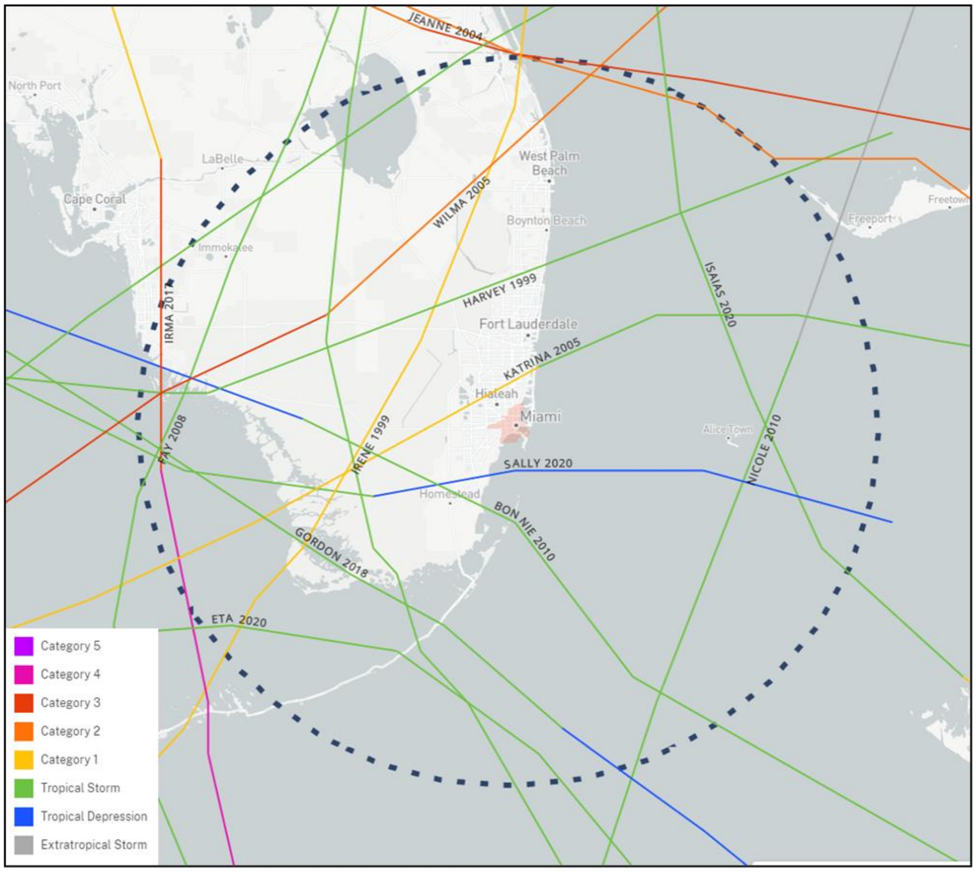
2.4 Major Storm Events

The Broad Causeway bridge is within a region prone to hurricane impact. According to NOAA's hurricane database (NOAA, 2024b), 18 storms of Category 1 strength or higher have passed within 90 nautical miles of the bridge location since 1950. **Figure 2-3** illustrates the paths of these 18 storm events, while **Table 2-2** lists their names and dates.

Table 2-2 Historical hurricanes passing within 90 nmi of the Broad Causeway bridge.

Name	*Dates*	*Name*	*Dates*
Nicole 2022	*Nov 06, 2022 to Nov 11, 2022*	*Andrew 1992*	*Aug 16, 1992 to Aug 28, 1992*
Irma 2017	*Aug 30, 2017 to Sep 13, 2017*	*Floyd 1989*	*Oct 09, 1989 to Oct 14, 1989*
Matthew 2016	*Sep 28, 2016 to Oct 10, 2016*	*David 1979*	*Aug 25, 1979 to Sep 08, 1979*
Wilma 2005	*Oct 15, 2005 to Oct 26, 2005*	*Inez 1966*	*Sep 21, 1966 to Oct 11, 1966*
Katrina 2005	*Aug 23, 2005 to Aug 31, 2005*	*Betsy 1965*	*Aug 23, 1965 to Sep 13, 1965*
Jeanne 2004	*Sep 13, 2004 to Sep 29, 2004*	*Isbell 1964*	*Oct 09, 1964 to Oct 16, 1964*
Frances 2004	*Aug 25, 2004 to Sep 10, 2004*	*Cleo 1964*	*Aug 20, 1964 to Sep 11, 1964*
Irene 1999	*Oct 12, 1999 to Oct 19, 1999*	*Donna 1960*	*Aug 29, 1960 to Sep 14, 1960*
Erin 1995	*Jul 31, 1995 to Aug 06, 1995*	*King 1950*	*Oct 13, 1950 to Oct 20, 1950*

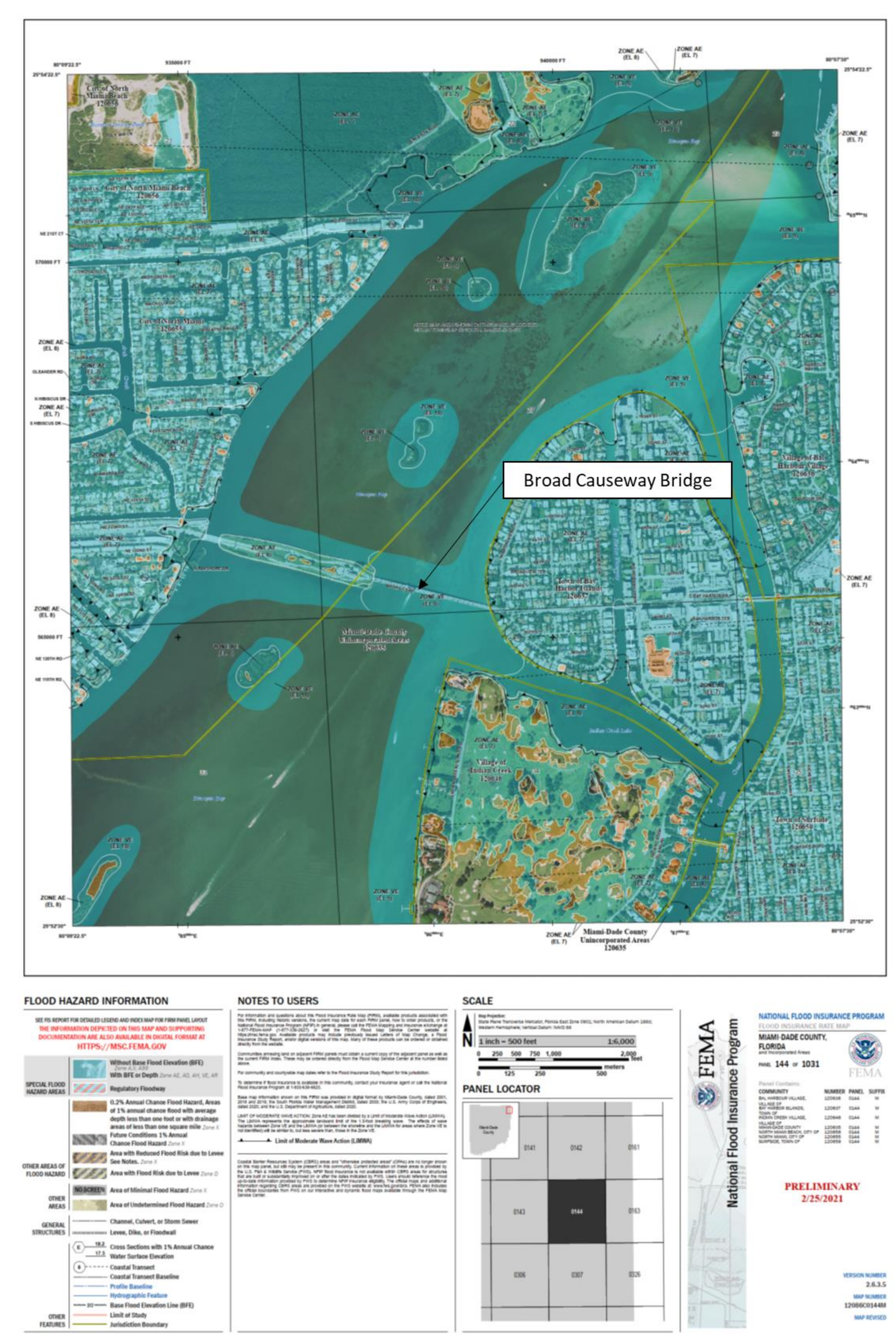
Figure 2-3 Historical hurricane tracks passing within 90 nmi of the Broad Causeway bridge (NOAA, 2024b).



2.5 Major Storm Events

The Federal Emergency Management Agency (FEMA) produces Flood Insurance Studies (FIS) and Flood Insurance Rate Maps (FIRMs) for flood-prone areas across the country. According to FIS 12086CV001B (FEMA, 2021), the 100-year stillwater elevation at a transect just to the east of the bridge location is +6.4 ft NAVD88. The 100-year wave crest elevation at the same location is +9.6 ft NAVD88. The bridge is located within FIRM (FEMA, 2021) 12086C0144M shown in **Figure 2-4** under a Zone VE, elevation +9 ft NAVD88 while the island in the center of the bridge is in Zone AE, elevation +8 ft NAVD88.

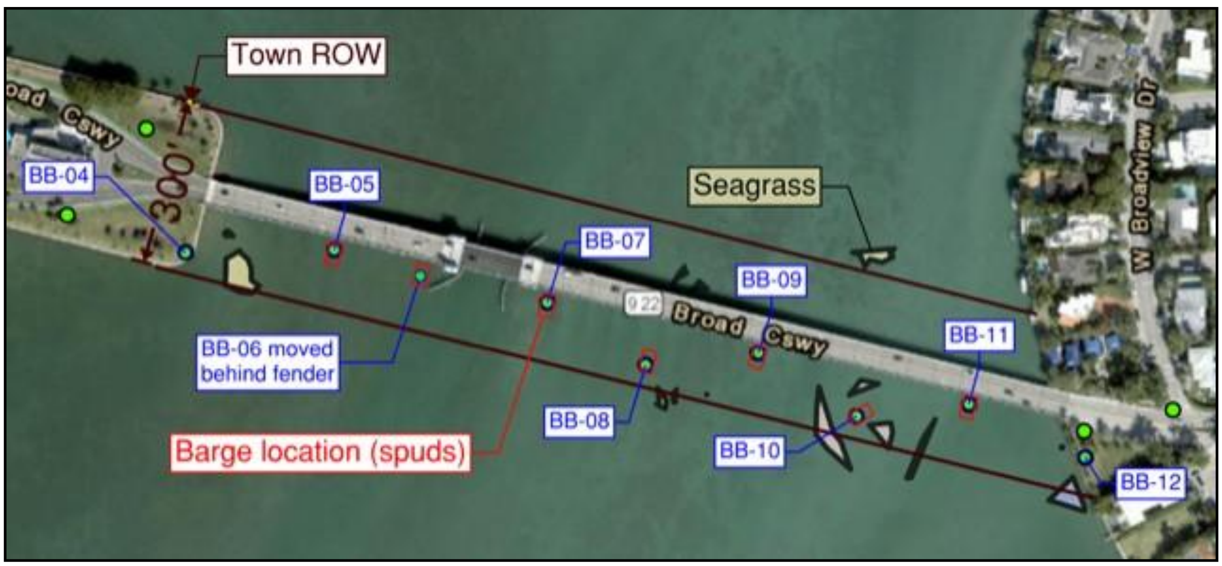
Figure 2-4 FEMA FIRM 12086C0144M.



2.6 Sediment Characteristics

Sediment information used in this analysis was obtained from site-specific sediment data provided by AREHNA Engineering as presented in the Preliminary Report of Geotechnical Exploration (Arehna, 2023). Grain size analysis from land boring BB-12 (12 to 14 feet below surface) nearest to the Broad Causeway bridge was selected in order to determine representative sediment characteristics that could reasonably be applied to the project site. **Figure 2-5** shows the location of the core boring relative to the Broad Causeway bridge. From this information, it was determined that the sediment composition is mostly sandy silt and silty sand with an average median grain size (D_{50}) value of 0.08mm. At the time of this report, borings BB-05 through BB-11 located in the ICWW have not been performed. The water boring information is expected to be available during final design and will be used in predicting scour for the future final design BHR.

Figure 2-5 Core boring location (BB-12) to determine the representative sediment characteristics for the Broad Causeway bridge.



2.7 Bathymetry and Topography Data

A numerical model grid was developed as part of this project to simulate water levels, waves, and currents at the Broad Causeway bridge (as described in **Section 3.0**). This model was developed using the best available topography and bathymetry data from the National Geodetic Survey (NGS) (NOAA, 2024d). The NOAA NCEI Continuously Updated Digital Elevation Model (CUDEM) bathymetric dataset constituted the main source of bathymetry data throughout the region, with a spatial resolution of 10 feet and a horizontal and vertical accuracy of 1.5 feet and 3 feet, respectively. After reviewing this existing model and its supporting data, it was determined that the bathymetry and topography data used to develop the model grid is the best available data for use in this analysis that spans the entire model domain. Based on this existing model data, the maximum depth noted through the main channel under the bridge is approximately -12.5 ft NAVD88. Note that during the ensuing design phase of this project, the model will be updated with site-specific hydrographic survey data as appropriate.

2.8 Site Investigation

Project team members performed a field visit to evaluate site conditions at the bridge location. **Figure 2-6** and **Figure 2-7** show photos of the existing Broad Causeway Bridge; however, note that the current design plan is to replace the existing bridge with an entirely new structure. In addition, the existing shore protection for the adjacent island on the west end of the bridges was reviewed and analyzed to determine its suitability to support the proposed roadway and drainage features. A memo documenting the results of the shore protection evaluation is provided in **Appendix A**.

Figure 2-6 Aerial photo of the Broad Causeway Bridge from the northeast side of the bridge looking toward the west.

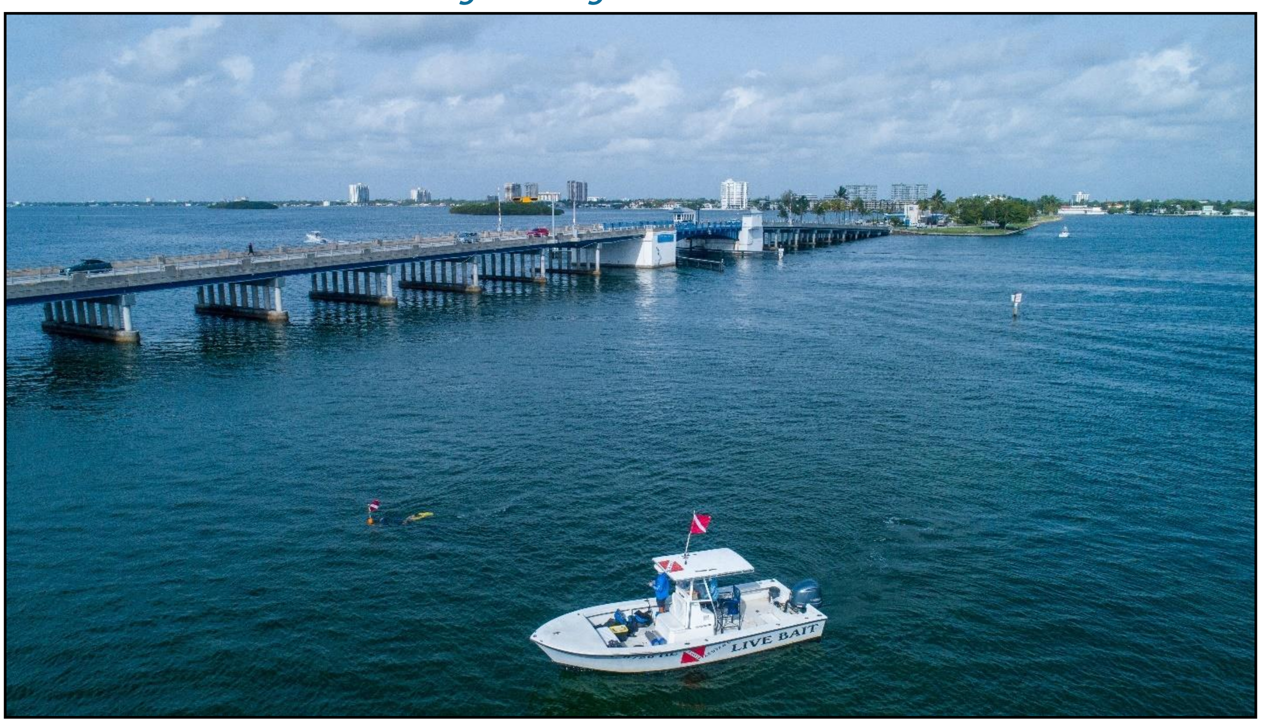


Figure 2-7 Aerial photo of the Broad Causeway Bridge from the southeast side of the bridge looking toward the east.

2.9 Proposed Bridge Geometry

The existing bridge layout consists of ten bridge piers spanning the Biscayne Bay (Piers 10 through 19). **Figure 2-8** below shows a plan view of the proposed bridge piers 12 through 14 (darker grey) overlaying the existing bridge design (light grey). Note that the proposed bridge is slightly south of the existing bridge. **Figure 2-9** shows the elevation view of the channelized portion of the proposed bridge pier corresponding to the plan view in **Figure 2-8**.

Figure 2-8 Layout of the bridge piling locations for the proposed bridge (dark grey) and the existing bridge (light grey)

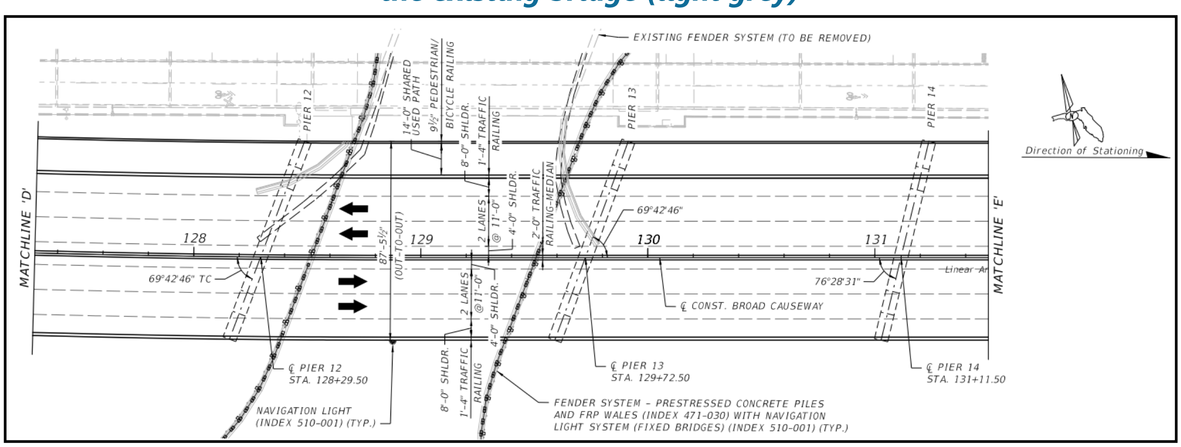
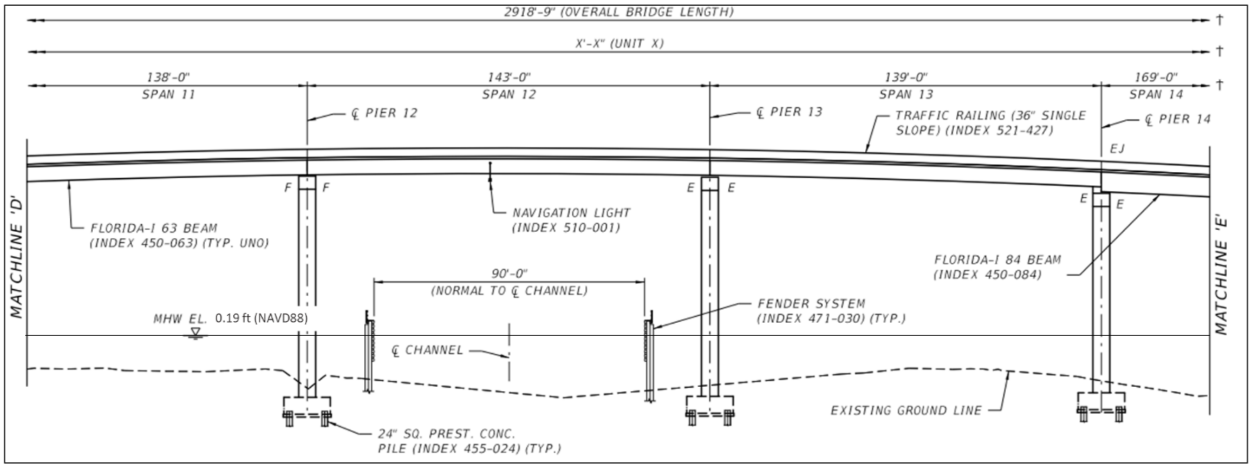


Figure 2-9 Cross-section view of the proposed bridge pile design



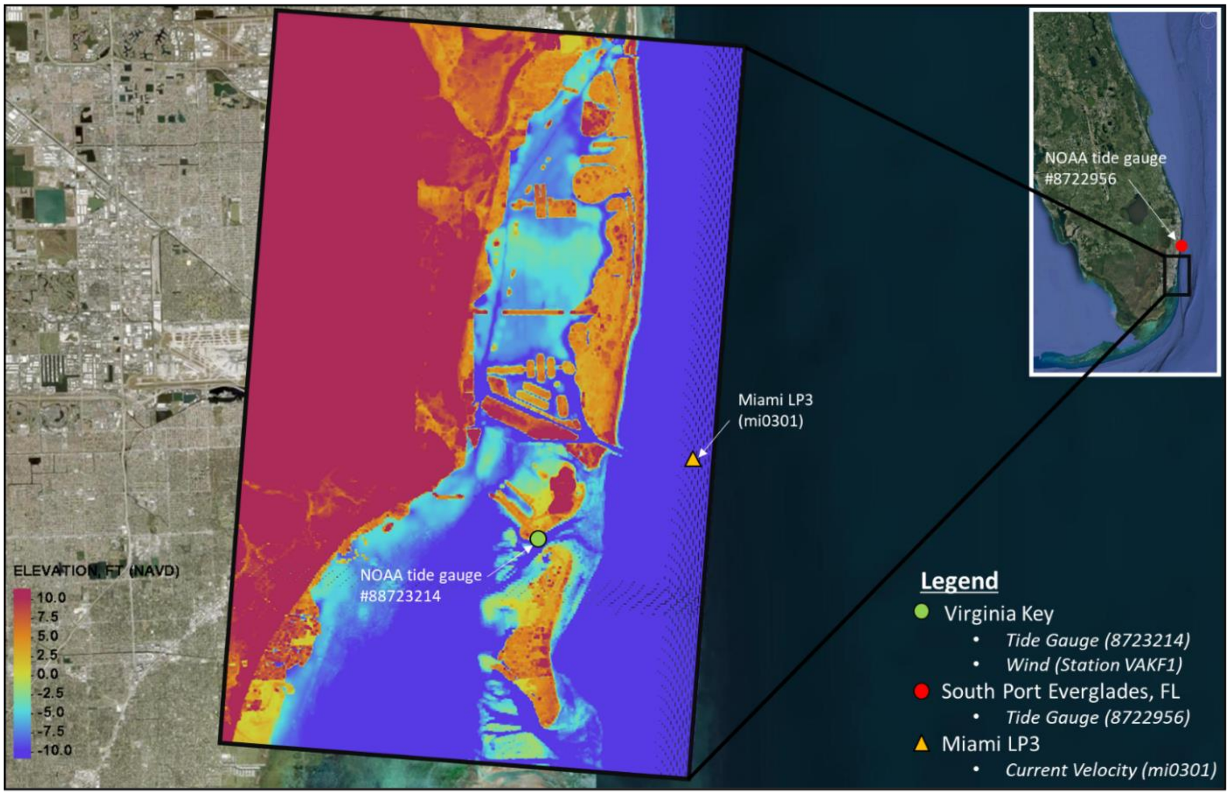
3.0 HYDRODYNAMIC AND WAVE MODEL

In order to determine the flow characteristics required for the scour calculations, a numerical model of the local and regional hydrodynamics and waves was developed. In this study, the Delft3D hydrodynamic model was used. Delft3D is a widely-used and validated numerical model which incorporates the effects off astronomic tides, wind, waves, and meteorological forces to simulate time-varying hydrodynamics in two or three dimensions (Deltares, 2011). In order to properly represent the physics of storm surge propagation and wave generation, the Delft3D model was dynamically coupled throughout the entire simulation using integrated FLOW and WAVE modules. Both models use the same unstructured mesh and information is continuously passed from one model to the other (e.g., water levels are passed from the FLOW to WAVE and wave radiation stresses are passed from WAVE to FLOW). This technology allows for a more accurate solution and is scalable to a wide range of modeling studies involving coastal/ocean circulation.

3.1 Delft3D Model Development

The base Delft3D model was developed using the bathymetry from the 2019 Continuously Updated Digital Elevation Model (CUDEM), which encompasses the entirety of Biscayne Bay. Horizontal and vertical coordinates are set to State Plane Florida West and NAVD88, respectively. **Figure 3-1** depicts the 17.5 miles by 11.5 miles model domain (black line) where a non-rectangular grid was developed with spatial resolution ranging from 400 feet offshore to 100 feet within Biscayne Bay. This scale model domain is necessary to represent the physics of storm surge and wave propagation from the deep ocean into the coastal floodplain.

Figure 3-1 Delft3D model bathymetry, existing conditions. The black solid line represents the model domain.



Accuracy of model results in the nearshore is largely dependent on the specification of accurate boundary conditions along the open ocean boundary. The model used data from the NOAA tide gauges (8723214 and 8722956) propagated offshore as water level boundary conditions. The Broad Causeway Bridge is located within the Biscayne Bay and protected by a barrier island, and hence, is primarily dominated by tidal forcing. A spatially uniform wind forcing was included in the model collected from the meteorological station (8723214) at a height of 28 feet, located at Virginia Key, approximately 10.8 miles from the Broad Causeway Bridge. All other boundaries are no-flow with essential tangential slip conditions.

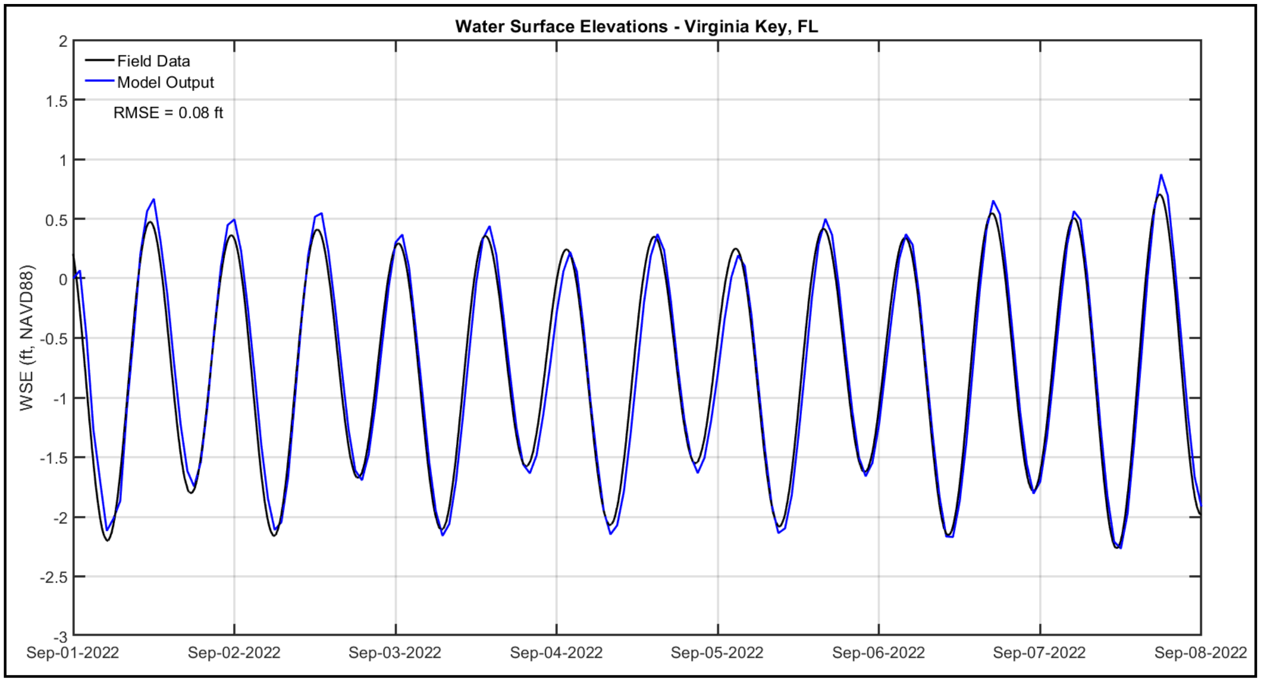
3.2 Model Validation

3.2.1 Tidal Simulation

The model was initially executed with the adjusted water level dataset from the nearest NOAA tide gauges alone to assess how well the modeled water levels compare with predicted water levels at NOAA Station 8723214 in Virginia Key. Looking at a tides-only comparison serves to validate the underlying bottom elevations and friction values in the model before introducing more complex storm surge and wave physics. The total simulation time for the tides-only scenario was 7-days from September 1st, 2022, to September 8th, 2022.

Figure 3-2 shows the comparison between modeled and predicted water levels over the 7-day period; the root-mean-square (RMS) error between the two time-series is 0.08 feet and the model is well-behaved with regard to astronomic tidal circulation.

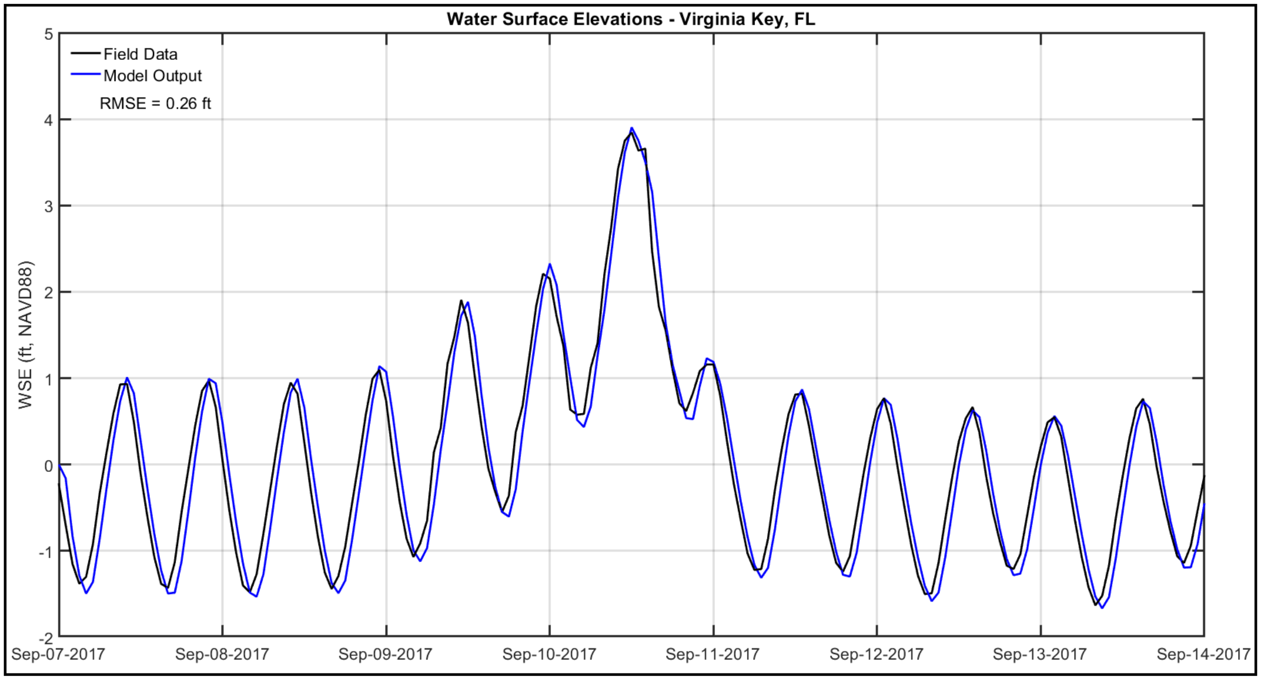
Figure 3-2 Modeled (blue) versus predicted (black) water levels in Virginia Key; tide only.



3.2.2 Hurricane Irma

Modeled water levels from the Delft3D simulation of Hurricane Irma (2017) were compared to the measured data at NOAA water level gauges in the vicinity of the Broad Causeway bridge location to assess the ability of the model to replicate the hydrodynamic conditions during the storm. Similarly, the model was executed with the adjusted water level dataset from the nearest NOAA tide gauge at Virginia Key, including the spatially constant and temporally variable wind conditions. A 7-day simulation period (September 7th, 2017 00:00 – September 14th 2017 00:00) was then performed and results were compared with measured data. Freshwater inflows were not included in the model setup, since the hydrodynamics are dominated by tidal and wind forcing rather than rainfall runoff. **Figure 3-3** illustrates the comparison between the measured and modeled water levels during Hurricane Irma at the Virginia Key gauge with an RMS error between the two time-series of 0.26 feet.

Figure 3-3 Modeled (blue) versus predicted (black) water levels in Virginia Key during Hurricane Irma.



3.3 Storm Simulations

After the modeled hydrodynamic conditions were verified against measured water levels, design hurricane storm surge hydrographs were included in the model to simulate the 50-, 100-, and 500-year return period events at the bridge location. The hurricane hydrographs were obtained from the Design Storm Surge Hydrographs database (Location 2302) for the Florida Coast prepared by the FDOT (FDOT, 2003) and adjusted using the 50-, 100-, and 500-year stillwater elevations from the most recent FEMA FIS report (FEMA, 2021) for the coastal transect closest to the Broad Causeway bridge (Transect 112). **Figure 3-4** illustrates the observation locations of the selected hurricane hydrographs and FIS still water elevations. **Table 3-1** presents the peak values of water level obtained from the FDOT and FIS reports in the vicinity of the bridge.

In addition, a statistical analysis was performed using the Generalized Pareto Distribution (GPD) on the wind dataset to determine the corresponding 50-, 100-, and 500-year wind magnitude (10 ft. above ground (MSL)) to be implemented in the storm simulations. For this case, the wind data from the Miami International Airport was used due to the length of data available at this meteorological station (~70 years). A 40-mph wind magnitude threshold and a 72-hour event interval were determined for this analysis. **Figure 3-5** presents the return period graphical representation for the corresponding wind dataset with model input values shown in **Table 3-1**.

Table 3-2 presents the water level and current velocity peaks at the bridge extracted from the numerical model along with the calculated significant wave crest elevation. **Figure 3-6** illustrates the peak water levels during the 100- year event; away from the bridge, water levels were higher on the southern end of the pass versus the north. **Figure 3-7** presents the peak current velocity magnitude for the 100-year event.

Table 3-1 Peak water level and wind magnitude model input for the 50-, 100-, and 500-year storm simulations.

Event	Peak Water Level (NAVD88)	Wind Magnitude and Direction Event	
	(ft)	(mph)	(Degrees)
50-Year	5.8	73	90
100-Year	6.4	79	90
500-Year	8.4	116	90

Table 3-2 Modeled peak water level, depth-averaged current magnitude and wave characteristics for the 50-, 100-, 500-year storm simulations at the bridge.

Event	Peak Water Level (NAVD88)	Depth-averaged current magnitude	Significant Wave Crest Elevation
	(ft)	(ft/s)	(ft)
50-Year	5.5	2.5	8.8
100-Year	6.5	1.6	9.6
500-Year	8.3	1.5	12.2

Figure 3-4 (Left) Location of the coastal transect #112 used to extract the FIS stillwater level for the 50-, 100-, and 500-year return period. (Right) Map showing the location (2302) of the extracted hurricane hydrographs.

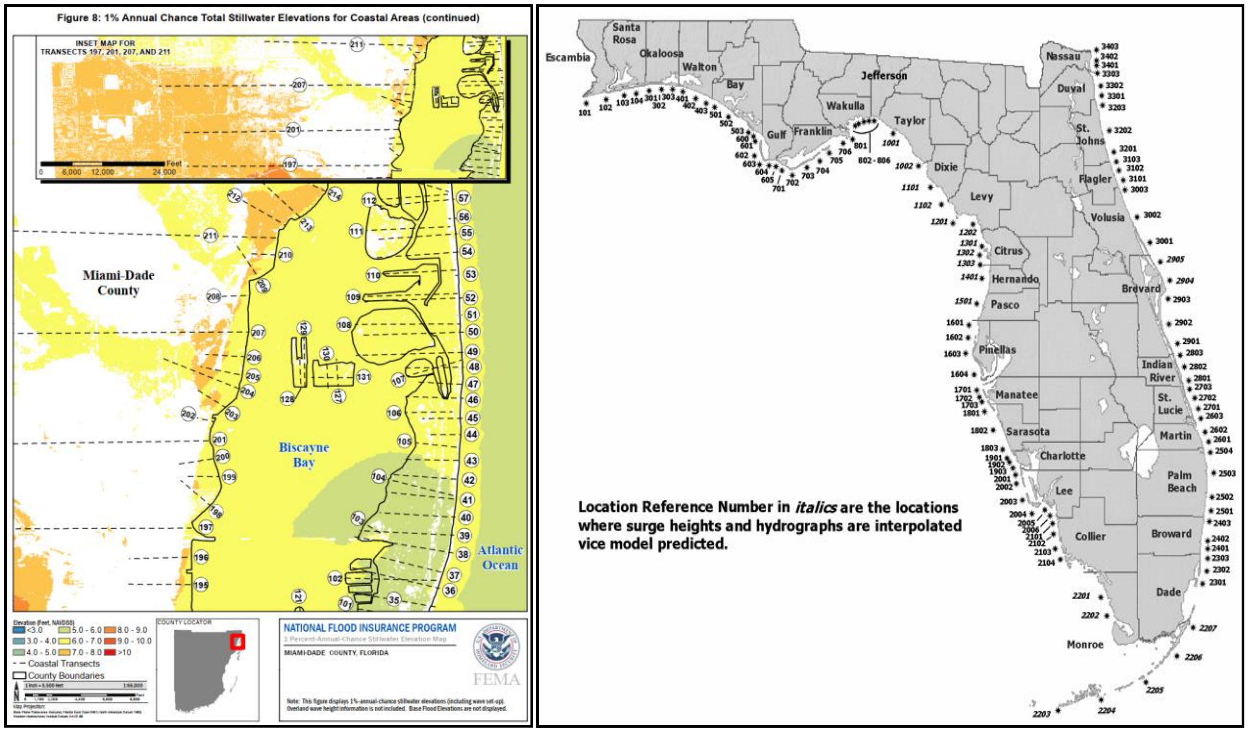


Figure 3-5 Return period of the wind magnitude using the Miami International Airport wind dataset from 1954.

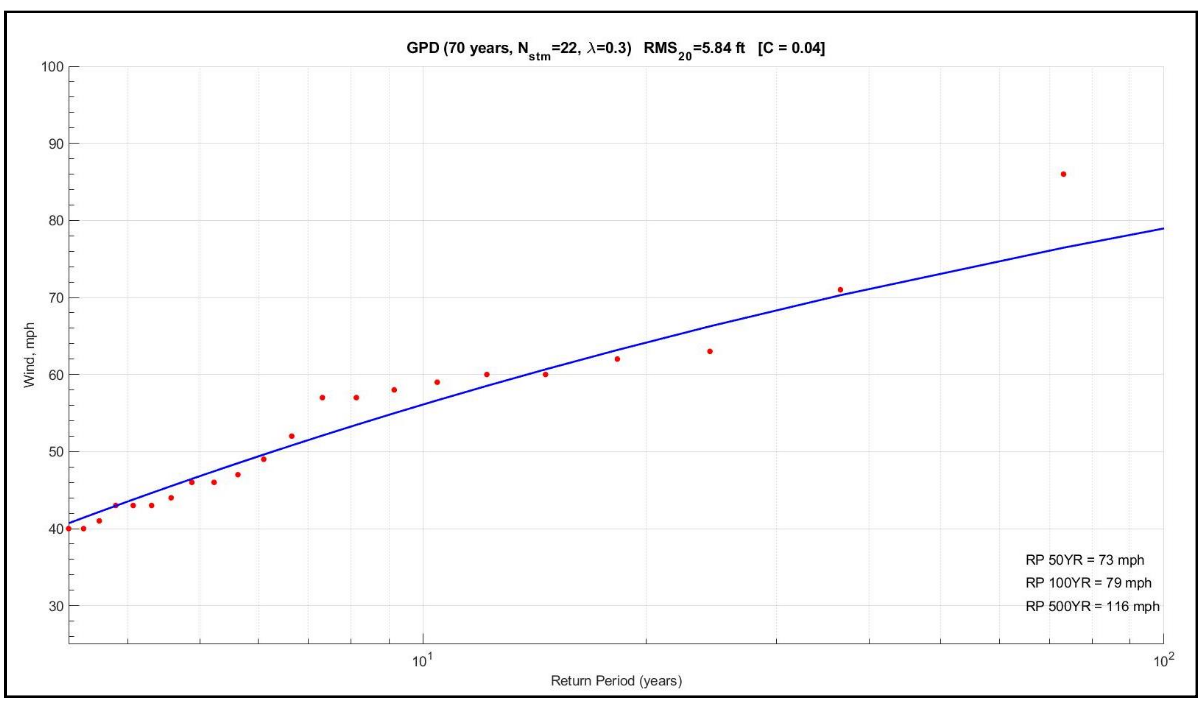


Figure 3-6 Peak water level during the 100-year event.

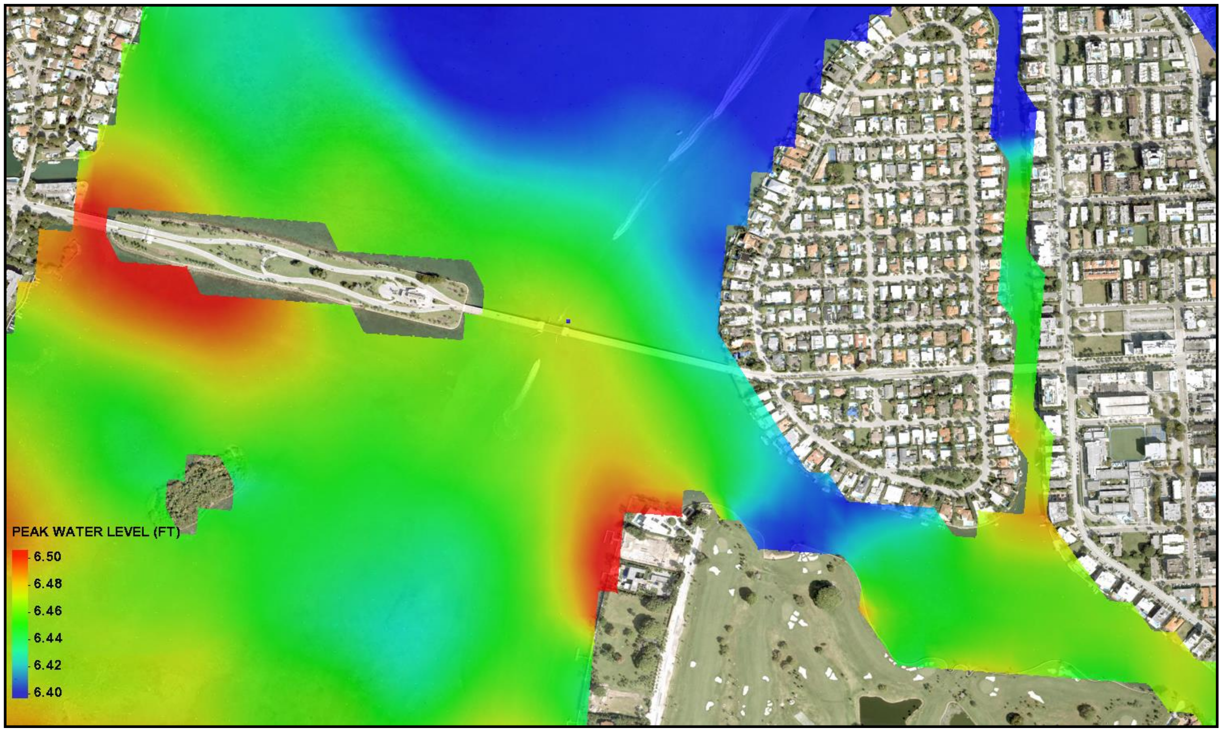
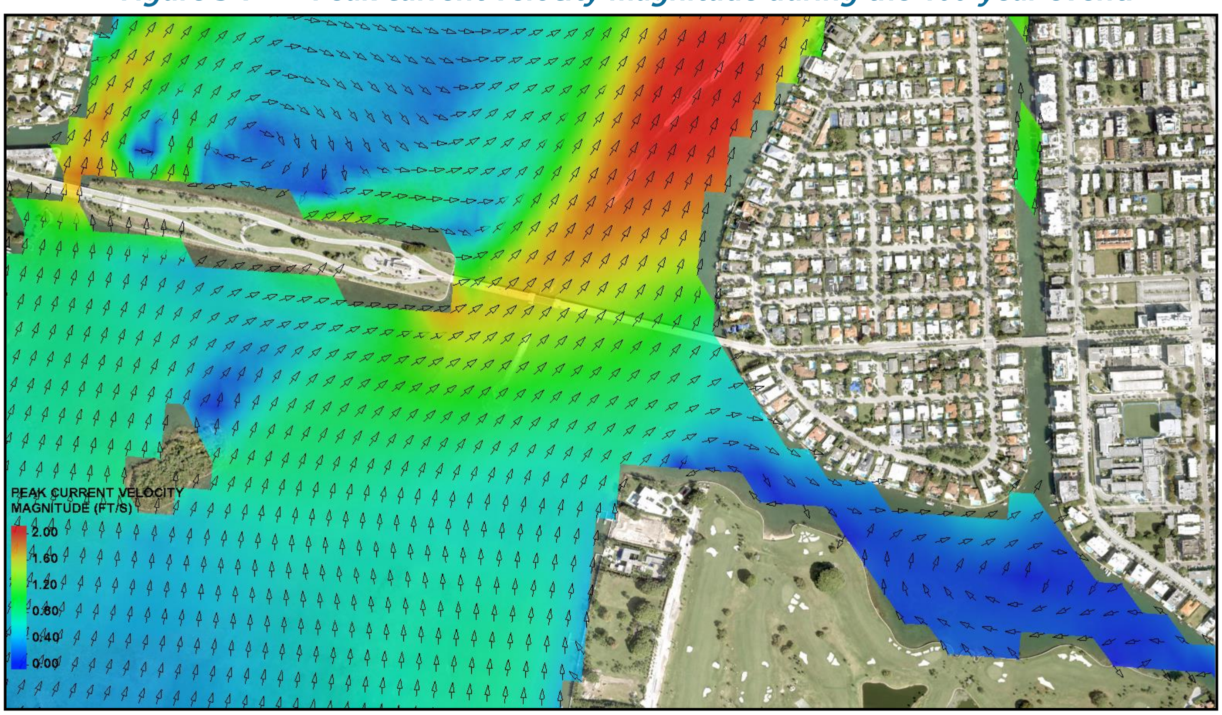


Figure 3-7 Peak current velocity magnitude during the 100-year event.



4.0 SCOUR, WAVE FORCES, AND DECK DRAINAGE

4.1 Scour

The model results produced in **Section 3.3** were applied to calculate scour conditions for the 50-, 100- and 500-year storm conditions. Four types of scour analysis are presented herein: long-term scour (**Section 4.1.1**), contraction scour (**Section 4.1.2**), local scour (**Section 4.1.3**), and abutment scour (**Section 4.1.4**). The results provided in this section will serve to support the preliminary design of the bridge pier foundations during the PD&E Phase; however, **the conceptual bridge pier sections provided in this PD&E study could change as the project progresses to final design. The scour calculations and results detailed below may change as a result, so the results presented herein are considered preliminary.**

4.1.1 Long-term Scour

At the time of this report, there is no known survey data for assessing the bathymetric changes across or along the channel spanned by the Broad Causeway bridge. However, it should be noted that most of the shoreline immediately adjacent to the bridge has some form of armoring (e.g., revetment or seawall), hence, shoreline recession is anticipated to be negligible.

4.1.2 Contraction Scour

Based on the methods described in HEC-18 (Arneson et al. 2012), the expected contraction scour at the bridge for the 50-year, 100-year and 500-year events was calculated for the case of a narrowing river or channel cross-section as shown in **Figure 4-1**. To determine whether upstream flows are moving sediment (live bed scour), HEC-18, recommends the equation below:

$$V_c = K_u y^{1/6} D^{1/3} \qquad (4-1)$$

where:

V_c = Critical velocity above which bed material of size D and smaller will transported, ft/s
y = Average depth of flow upstream of the bridge, ft
D = Particle size for V_c, ft
D_{50} = Particle size in a mixture of which 50 percent are smaller, ft
K_u = 11.17 English units

Figure 4-1 HEC-18, Contraction Scour: Bridge Abutments set back from channel

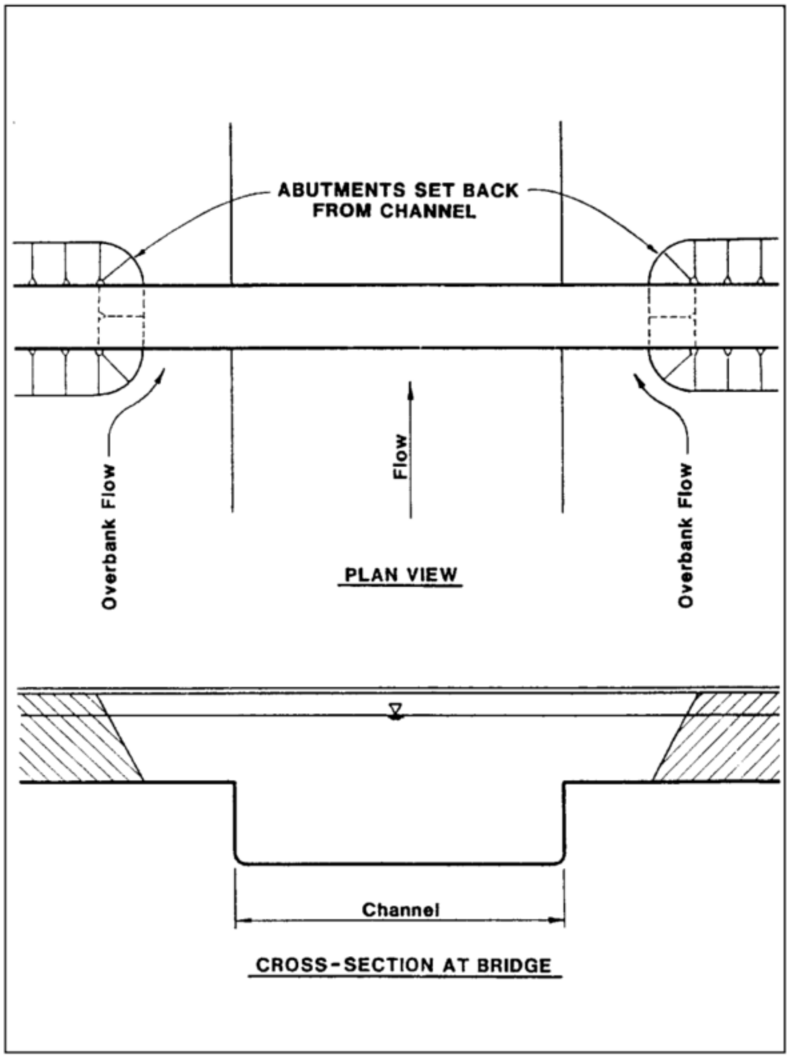


Table 4-1 presents the parameters used for the contraction scour analysis extracted from the numerical model at the location shown in **Figure 4-2**. Based on **Equation 4-1**, a live bed flow regime is expected upstream of the site for all design storms. Since water flows through the bridge from north and south direction, the analysis was performed using both sides of the bridge to determine which direction of the flow produces the worst scour depths. HEC-18 recommends the use of Larsen's Live Bed Equation, shown as **Equation 4-2** but also warns that, "Lauren's equation will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of a natural contraction or if the contraction is the result of the bridge abutments and piers. At this time, however, it is the best equation available.":

$$\frac{y_2}{y_1} = (\frac{Q_2}{Q_1})^{6/7}(\frac{W_1}{W_2})^{k_1} \quad \textbf{(4-2)}$$

$y_s = y_2 - y_0$ (average contraction scour depth)

where:
y_1 = Average depth in the upstream main channel, ft
y_2 = Average depth in the contracted section, ft
y_0 = Existing depth in the contracted section before scout, ft
Q_1 = Flow in the upstream channel transporting sediment, ft^3/s
Q_2 = Flow in the contracted channel, ft^3/s
W_1 = Bottom width of upstream main channel transporting bed material, ft
W_2 = Bottom width of main channel in contracted section less pier width(s), ft
k_1 = Exponent determined below

V^*/T	k_1	*Mode of Bed Material Transport*
<0.50	*0.59*	*Mostly contact bed material discharge*
0.5 to 2.0	*0.64*	*Some suspended bed material discharge*
>2.0	*0.69*	*Mostly suspended bed material discharge*

V^* = $(\vartheta_0/\Delta)^{1/2}$ = $(gy_1S_1)^{1/2}$ shear velocity in the upstream section, ft/s
T = Fall velocity of bed material based on the D50, ft/s
g = Acceleration of gravity (32.2 ft/s^2)
S_1 = Slope of energy grade line of main channel, ft/ft
ϑ_0 = Shear stress on the bed, (lb/ft^2)
Δ = Density of water (1.94 slugs/ft^3)

Figure 4-2 Model observation locations for contraction scour calculations.

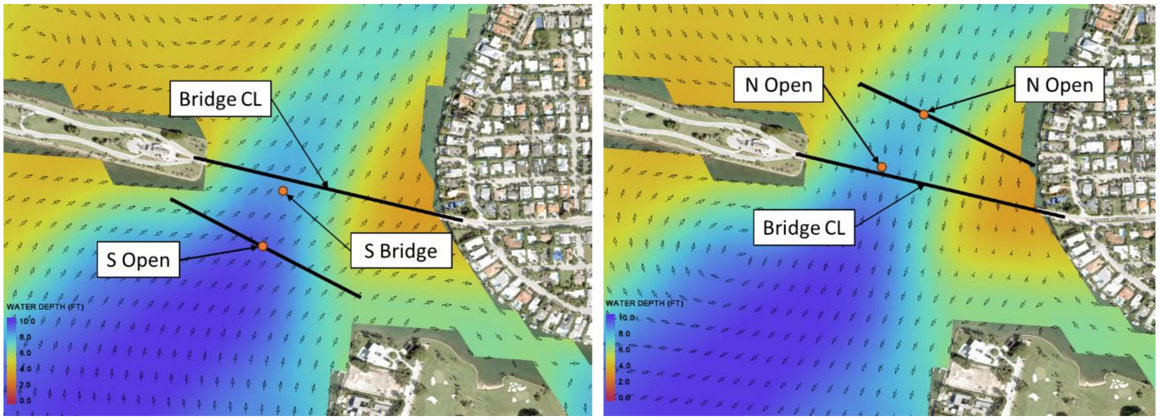


Table 4-1 Summary of contraction scour parameters and results.

Name	50-year		100-year		500-year	
Current from:	N Open	S Open	N Open	S Open	N Open	S Open
D50 (mm)	0.08		0.08		0.08	
k_1	0.69		0.69		0.69	
Initial depth at bridge (ft)	13.7	14.5	14.8	15.5	16.5	17.4
Channel width at bridge* (ft)	630	630	630	630	630	630
Depth-averaged velocity at bridge (ft/s)	2.5	2.2	1.6	1.6	1.5	1.3
Flow rate at bridge (cfs)	21,577.5	23,127.5	14,918.4	17,980.0	15,592.5	16,399.5
Upstream depth (ft)	12.8	15.5	13.9	16.5	15.7	18.3
Upstream channel width (ft)	1,239	1,364	1,239	1,364	1,239	1,364
Upstream depth-averaged velocity (ft/s)	2.7	1.7	1.9	1.3	1.6	0.9
Upstream flow rate (cfs)	42,653	35,928	31,746	29,247	31,025	21,209
Scour type	live-bed	live-bed	live-bed	live-bed	live-bed	live-bed
Scour depth below initial bed (ft)	-2.3	1.6	-3.4	1.0	-2.6	0.4
Design scour depth (ft)	1.6		1.0		0.4	

*The total width of the piers was removed from the channel width at the bridge

Preliminary pile dimensions were used to estimate the amount of obstruction at the bridge location in addition to the channel narrowing (estimated to be 9.0%). The 'upstream' channel widths were taken to be the width of the main Biscayne Bay channel at the aforementioned observation locations, and the bottom depth was determined by the bathymetric survey available. **Table 4-1** presents the results of the contraction scour estimates, which range from 0.4 ft for the 500-year event to 1.6 ft for the 50-year event. The 500-year current velocity and scour is less than the 50-year event likely due to the increased water levels for the 500-year event creating less constricted flow through the bridge opening. **Note that these results could change if the bridge pile dimensions are modified for the final design phase of the project.**

4.1.3 Local Scour

Local scour is defined as the erosion of bed sediment around flow obstacles, such as bridge piers and abutments. In the preliminary bridge design, there will be three to four support piers in the Biscayne Bay. The methodology outlined in Section 2.5 of the FDOT Bridge Scour Manual (FDOT 2022) and included in the FDOT Complex Pier Local Scour Calculator spreadsheet tool was used to compute local scour for the 50-year, 100-year and 500-year events based on the largest modeled flow at the bridge location and the most conservative (largest) potential pile design. Because the exact locations and bottom elevations are not defined at this time, the bottom depth was extracted from the bathymetric dataset available. **Figure 4-3** illustrates the ICWW potential pier cross-section in the preliminary design.

Table 4-2 summarizes the parameters used in the local scour calculations. The local pile scour was calculated to be 6.7 ft below the initial bed level for the 50- year event, 6.2 ft below the initial bed for the 100-year event and 6.2 ft below the initial bed for the 500-year event. **Note that these results could change if the bridge pile dimensions are modified for the final design phase of the project.**

Figure 4-3 Cross-section of potential pier in preliminary design.

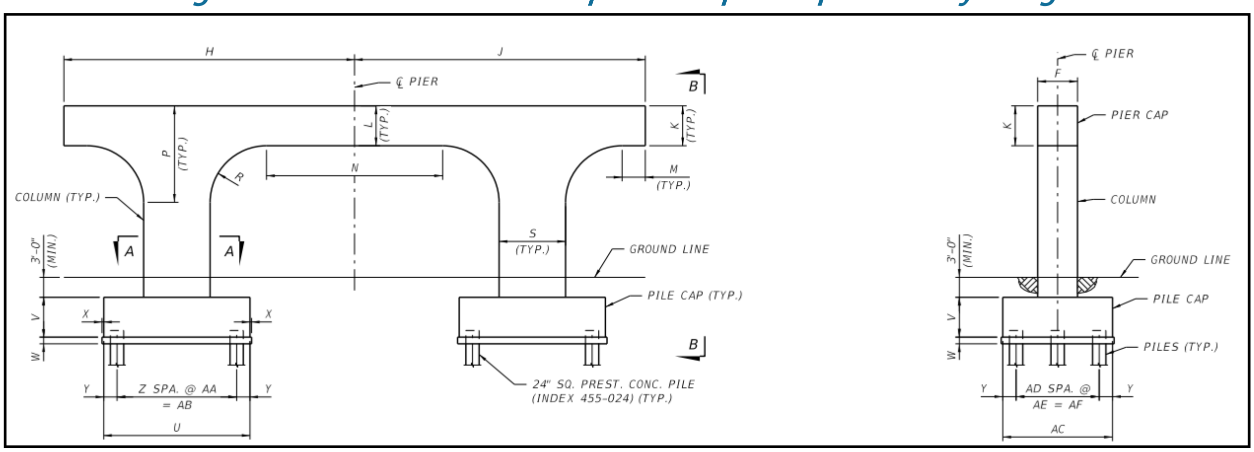


Table 4-2 Summary of parameters in local scour calculations.

Parameter	50-year	100-year	500-year
D50 (mm)	0.1	0.1	0.1
Initial depth at bridge (ft)	14.1	15.2	17.0
Pile diameter perpendicular to flow (ft)	2.0	2.0	2.0
Effective pile diameter (ft)	6.4	6.6	6.6
Depth-averaged velocity at bridge (ft/s)*	2.5	1.6	1.5
Local pile scour depth below initial bed (ft)	6.7	6.2	6.2

Angle of attack of 30 degrees

4.1.4 Abutment Scour

Abutments will be armored with seawall and rubble at the toe. Hence, abutment scour will be dismissible due to this proposed toe protection.

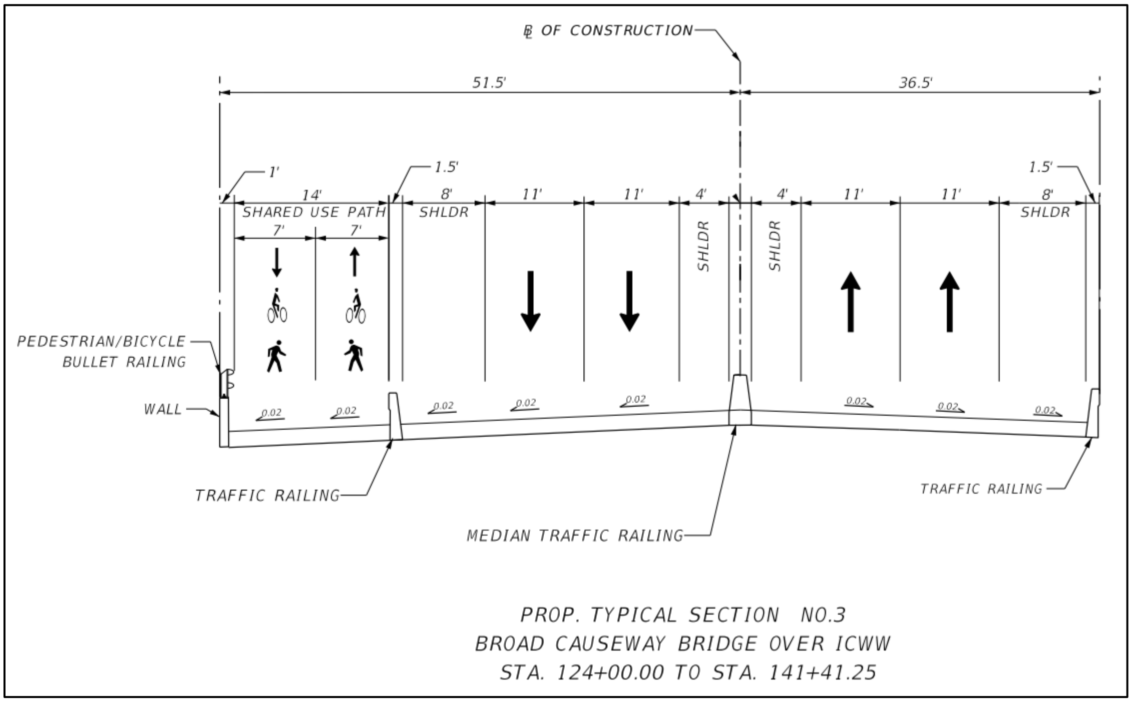
4.2 Wave Forces on the Proposed Bridge Superstructure

The Broad Causeway Bridge is located within the Biscayne Bay and protected by a barrier island, and hence, is primarily dominated by tidal forcing. However, wind-induced waves can be generated during moderate to extreme atmospheric conditions. Based on results presented in **Table 3-2**, the 100-year wave crest elevation is below the proposed bridge superstructure elevation of approximately 15 ft NAVD88 (close to the end of the bridge), and wave forces may be dismissed.

4.3 Bridge Deck Drainage

The proposed preliminary bridge typical section and profile are shown below:

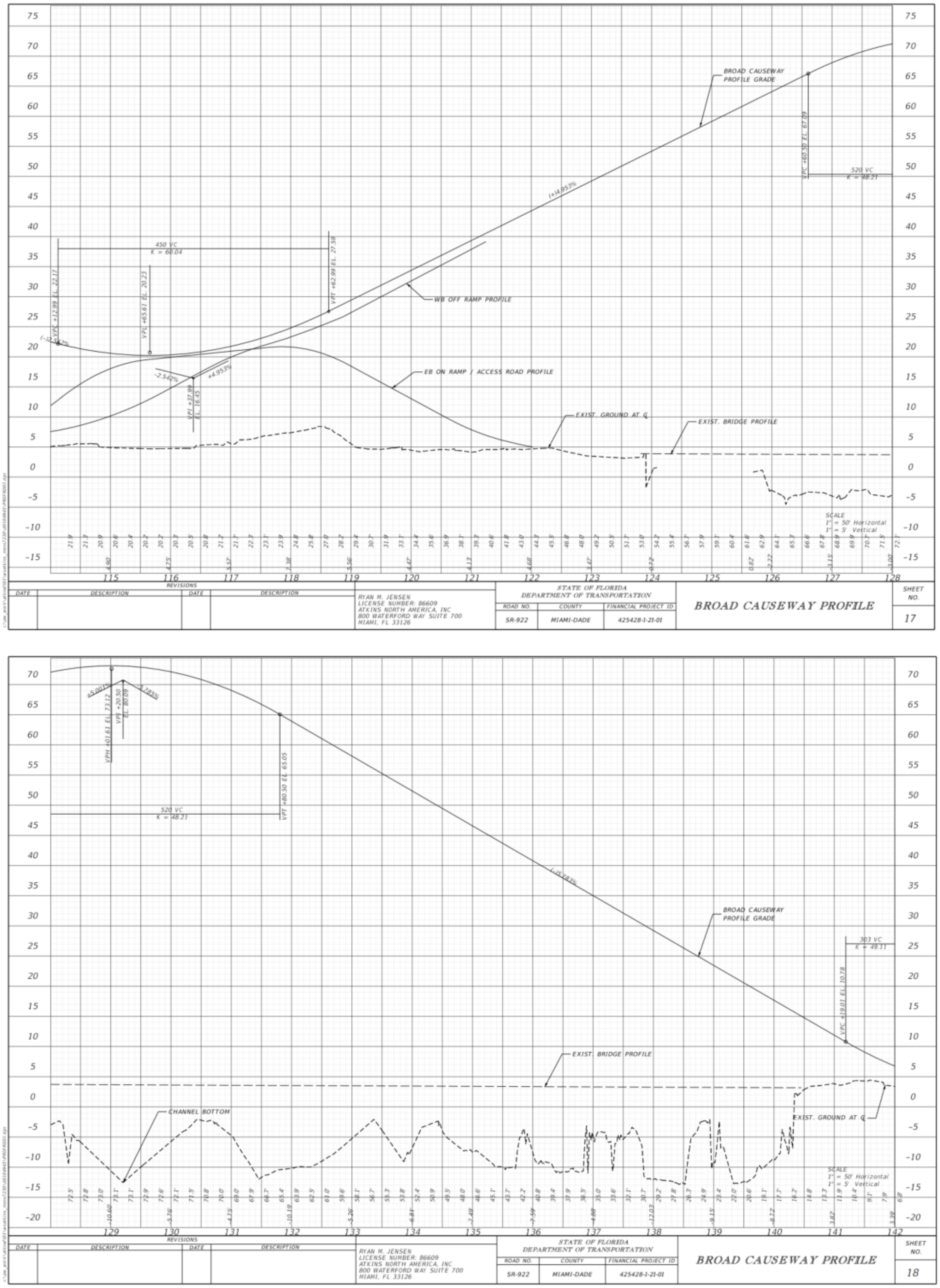
Figure 4-4 Proposed typical section on Broad Causeway Bridge



The cross slope of the bridge will be 2%, crowned at the barrier wall between the 4, 11-ft, travel lanes. Therefore, a width of 36.5 ft. of deck will accumulate against the outer barrier walls.

From the Concept Plans, as shown in **Figure 4-5** below, the western slope of the bridge, from sta 124+00 – 129+02, will have a 4.935% grade, while the eastern bridge slope, from 129+02 to 141+41, will have a 5.783% grade:

Figure 4-5 Preliminary bridge slopes from concept plans



For arterials and collectors, a full width shoulder is defined in the FDM, Table 210.4.1, as 10 ft. in width:

Figure 4-6 FDM reference for full shoulder widths

Topic #625-000-002
FDOT Design Manual January 1, 2024

Table 210.4.1 Standard Shoulder Widths

Lane Type	# Lanes (One Direction)	Without Shoulder Gutter				With Shoulder Gutter			
		Outside		Median Or Left		Outside		Median Or Left	
		Full Width (feet)	Paved Width (feet)	Full Width (feet)	Paved Width (feet)	Full Width (feet)	Paved Width (feet)	Full Width (feet)	Paved Width (feet)
Travel Lanes	4-Lanes or more	10	5	10	4	15.5	8	15.5	8
	3-Lanes	10	5	10	4	15.5	8	15.5	8
	1-Lane & 2-Lanes	10	5	8	4	15.5	8	13.5	6
Aux. Lanes	ALL	10	5	8	4	11.5	4	11.5	4

Notes:

Without shoulder gutter:

(1) Consider 12-foot outside full width shoulder adjacent to travel lanes with high AADT or greater than 10% trucks.
(2) Consider providing a minimum 10-foot median shoulder where continuous barrier or guardrail is present.
(3) Outside shoulder widths for auxiliary lanes typically match those of the adjacent roadway; however, width may be reduced to 6-foot shoulder with 2-foot paved for right turn lanes when a bicycle keyhole is present.
(4) Pave the entire width of shoulders adjacent to concrete barriers. See ***FDM 215.4.6.1.***
(5) For RRR Projects:
 (a) an existing full width shoulder of 6-foot or greater may be retained
 (b) the following minimum existing outside paved shoulder widths may also be retained:
  i. 4-foot adjacent to travel lane
  ii. 2-foot adjacent to auxiliary lane
 (c) an existing unpaved median or left shoulder may be retained. Consider providing a 4-foot median or left paved shoulder adjacent to travel and auxiliary lanes where there are documented safety or maintenance concerns.

With shoulder gutter:

(1) Paved shoulders less than 6 feet in width with adjoining shoulder gutter must be the same type, depth, and cross slope as the roadway pavement.
(2) Shoulders must extend 4 feet beyond the back of shoulder gutter and have a 0.06 cross slope back toward the gutter.
(3) Required shoulder widths for auxiliary lanes typically match those of the adjacent roadway.

210 – Arterials and Collectors

With a design speed of 30 mph and an 8 ft. shoulder (non-full width), the FDOT Drainage Manual, Section 3.9.1, sets the required limits of the spread of the runoff at keeping ½ the outside travel lane clear, as shown below:

Figure 4-7 2024 FDOT Drainage Manual Spread Criteria

3.9.1 Spread Criteria

The spread criteria listed is for permanent design and temporary construction conditions. Limit the spread resulting from a rainfall intensity of 4.0 inches per hour as follows.

Table 3.5: Spread Criteria

Typical Section Condition	Design Speed* (mph)	Spread Criteria**
Parking Lane or Full Width Shoulders	All	No encroachment into the lane
Left Turn Lanes	Design Speed > 45	Keep 8' of lane clear
Right Turn Lanes	All	Keep ½ of lane clear
All Other	Design speed ≤ 45	Keep ½ of lane clear
	45 < Design Speed ≤ 55	Keep 8' of lane clear
	Design Speed > 55	No encroachment into the lane
Limited Access (Including Ramps)	All	No encroachment into the lane.

* Use the work zone speed shown in the Temporary Traffic Control Plans for temporary conditions. For more information on work zone speed, see FDM 240.

** The criteria in this column apply to travel, turn, or auxiliary lanes adjacent to barrier wall or curb, in normal or super-elevated sections.

The FDOT Drainage Design Guide, Section 6.3.2, states,

Figure 4-8 FDOT 2024 Drainage Design Guide, Section 6.3.2

Use the integrated form of Manning's equation to calculate spread in gutters.

$$Q = \frac{0.56}{n} S_x^{5/3} S_L^{1/2} T^{8/3}$$

where:
Q = Gutter flow rate, in cubic feet per second (cfs)
n = Manning's roughness coefficient (see Table B-2, Appendix B)
S_x = Pavement Cross Slope, in feet per feet (ft/ft)
S_L = Longitudinal Slope, in feet per feet (ft/ft)
T = Spread, in feet (ft)

Rearranging the gutter flow equation to solve for the spread:

$$Spread\ (ft) = \left[\frac{Q\,n}{0.56\,S_x^{5/3}\,S^{1/2}}\right]^{3/8} \qquad (4\text{-}3)$$

where,

$Q = Flow\ (cfs)$
$n = Manning's\ n$
$S_x = Cross\ Slope\ (ft/ft)$
$S = Longitudinal\ Slope\ (ft/ft)$

The inputs to the spread computations and spread results are shown in the tables below:

Table 4-3 Inputs to spread computations

Side	Width (ft)	Begin Sta	End Sta	Length (ft)	Flow (cfs)	Slope (ft/ft)	Sx (ft/ft)
East	36.5	12902	14140	1238	3.94	0.05783	0.02
West	36.5	10972	12902	1930	6.15	0.04953	0.02
Ramps	20	11568	12125	557	0.97	0.04953	0.02

Table 4-4 Results of Spread Computations

Location on Bridge	Spread (ft)	Allowable Spread (ft)
East Slope Spread	8.7	14
West Slope Spread	10.6	14
West Slope on Ramps	11.1	12.5

Since the spread on the bridge is less than ½ of the travel lane width, no bridge deck drainage will be needed. Bridge runoff will be conveyed to the roadway drainage system for collection.

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusions

A hydrodynamic and wave modeling study is presented in this report for the purposes of determining scour at the proposed bridge piers associated with the Broad Causeway bridge project. The project site is located at the ICWW between Bay Harbor Islands and the Florida mainland. A Delft3D hydrodynamic model was used to simulate the propagation of storm surge and waves from the deep ocean to the project site. Hurricane Irma is the storm of record for this area, and the wind and water levels associated with it were used to validate the model's performance compared to gauge data located near the Broad Causeway bridge.

Additionally, the FDOT storm surge hydrographs and FIS 50-, 100-, and 500-year stillwater elevations were scaled to represent various return period storm events (50- year, 100-year, and 500-year return periods). Using the model results from these storm simulations, the contraction scour and local scour were then computed at the proposed bridge piers. **Table 5-1** presents a summary of the scour results at the Broad Causeway Bridge. The contraction scour was calculated to be 1.0 ft and 0.4 ft for the 100-year and 500-year storm events, respectively. Similarly, the local scour was calculated to be 6.2 ft and 6.2 ft for the 100-year and 500-year storm events, respectively. This information can be used to help guide the planning process for the proposed Broad Causeway bridge project.

Table 5-1 Summary of calculated scour depths at the Broad Causeway Bridge Location.

Type of Scour	50-year Scour Depth (ft)	100-year Scour Depth (ft)	500-yr Scour Depth (ft)
Long-term	0	0	0
Contraction	1.6	1.0	0.4
Local	6.7	6.2	6.2
Total Scour	8.3	7.2	6.6
Scour Elevation	-16.6	-15.5	-14.9

**Based on existing channel elevation of -8.3 ft (NAVD88).*

5.2 Recommendations

The analysis provided herein is considered preliminary and is to be used only for planning purposes associated with the PD&E study. The list below details critical site-specific data that will need to be collected prior to final design in order for the scour analysis to be complete. Each of the items below is deemed necessary for the project site in accordance with standard engineering practice.

- Water level, current, and wave data needs to be collected at the site for at least a two-week period in order to validate the Deltf3D hydrodynamic model to local conditions.
- Sediment data (via soil borings or another acceptable method) needs to be collected and analyzed by a licensed geotechnical engineer at each of the proposed pier locations so that grain size distribution and median grain size can be determined.
- The final design of the bridge piers needs to be provided so that the scour calculations can be refined.

6.0 REFERENCES

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7.0 APPENDICES

7.1 APPENDIX A – MEMORANDUM: REVETMENT EXISTING CONDITIONS

January 17, 2024

Broad Causeway in Miami-Dade County, Florida – Revetment Existing Conditions

AtkinsRéalis was tasked with evaluating the existing conditions of a stone revetment located along a portion of Broad Causeway in Miami-Dade County, Florida. The area of interest included the portion of Broad Causeway that crosses Biscayne Bay, connecting the Bay Harbor Islands to the Keystone Islands. The existing revetment runs parallel to Broad Causeway and provides protection from wave action and erosion from Biscayne Bay. The following summarizes the revetment existing conditions, including field observations and an evaluation of the existing stone size.

Field Observations:

AtkinsRéalis performed a site visit on December 1, 2023, to document the existing conditions of the north and south revetments located along the Broad Causeway Bridge, as seen in Figure 1. Based on the site evaluation, the existing revetment appears to be composed of limestone ranging from one (1) to four (4) feet in diameter, as seen in Figure 2. The northern and southern ends of the revetments tie into an existing seawall, as shown in Figure 3. There appeared to be some stone displacement due to wave action, resulting in larger stone sizes being located along the revetment crest and smaller stones toward the structure toe.

The revetment slopes varied and appeared to be steeper on both ends (where it tied into the existing seawalls), compared to the remainder of the structure. The average width of each revetment was measured to be approximately 17 feet. The shortest distance from the landward edge of the revetment to the nearest structure was approximately seven (7) feet on the south revetment (measured to roadway railing) and approximately 30 feet on the north revetment (measured to roadway). Four (4) 18-inch reinforced concrete pipes (RCP) were located along the revetments as presented in Figure 4, with half of them damaged. Some areas of the revetment also showed evidence of erosion and scarping near the structure crest, as seen in Figures 5 and 6. Despite evidence of some stone displacement and erosion, the existing revetments appear to be in stable condition.

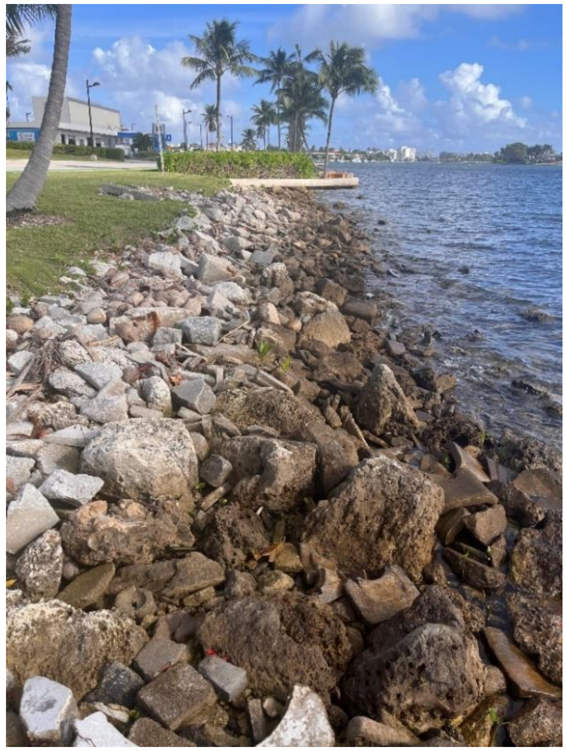



Figure 1: North (left) and south (right) existing revetments facing east.


Figure 2: Samples of the revetment limestone with size ranging from 1 to 4 feet in diameter.

Figure 3: South revetment at north (left) and south (right) tie-in to existing seawall. The north revetment has the same configuration.

Figure 4: Discharge outfalls located along both existing revetments. There are two discharge outfall structures on each existing revetment.

Figure 5: Erosion observed at both the north (left) and south (right) revetments, close to the discharge outfalls.

Figure 6: Erosion observed at both the north (left) and south (right) revetments, close to the discharge outfalls.

Evaluation of Existing Stone Size:

AtkinsRéalis performed a stone sizing analysis to determine the minimum armor stone size required for various return period events. Four (4) design scenarios were evaluated, including the 10-year, 20-year, 50-year, and 100-year return period events. The stillwater level, wave height, and wave period associated with each design scenario was determined based on the U.S. Army Corps of Engineers (USACE) Coastal Hazards System (V2.0), South Atlantic Coastal Study, which included analysis for Biscayne Bay, as summarized in Table 1 and Table 2.

Table 1 – North revetment return period parameters.

Design Scenario	Wave Height, feet	Wave Period, seconds	Stillwater Elevation, feet NAVD88
10-year Return Period	2.31	2.84	4.18
20-year Return Period	2.79	2.97	4.88
50-year Return Period	3.29	3.07	5.65
100-year Return Period	3.59	3.13	6.17

Table 2 – South revetment return period parameters.

Design Scenario	Wave Height, feet	Wave Period, seconds	Stillwater Elevation, feet NAVD88
10-year Return Period	2.51	3.01	4.17
20-year Return Period	3.02	3.15	4.89
50-year Return Period	3.55	3.28	5.67
100-year Return Period	3.87	3.35	6.20

The stone specific weight was assumed to be 145 pounds per cubic foot (pcf), which is a conservative estimate for the unit weight of limestone. Although the structure crest elevations, toe elevations, and structure slopes varied across the shoreline, these parameters were estimated based on aerial imagery and the 2019 topographic and bathymetric survey data available through the National Oceanic and Atmospheric Administration (NOAA) National Geodetic Survey (NGS). The north revetment was estimated to have a crest elevation of +2.0 feet referenced to the North American Vertical Datum of 1988 (NAVD88) and a toe elevation of -4.0 feet NAVD88. The south revetment was estimated to have a crest elevation of +3.0 feet NAVD88 and a toe elevation of -3.0 feet NAVD88. Both the north and south revetments were conservatively assumed to have a structure slope of 3H:1V. Based on the design wave parameters and estimated structure parameters, Table 3 and Table 4 summarize the minimum median armor stone weight and diameter required for each design scenario.

Table 3 – Stone sizing analysis results for the north revetment.

Design Scenario	Minimum Stone Median Weight (W_{50}), pounds	Minimum Stone Median Diameter (D_{50}), feet
10-year Return Period	153	1.02
20-year Return Period	259	1.21
50-year Return Period	425	1.43
100-year Return Period	552	1.56

Table 4 – Stone sizing analysis results for the south revetment.

Design Scenario	Minimum Stone Median Weight (W_{50}), pounds	Minimum Stone Median Diameter (D_{50}), feet
10-year Return Period	202	1.12
20-year Return Period	329	1.31
50-year Return Period	534	1.54
100-year Return Period	691	1.68

Overall, based on the above parameter estimates, the existing north and south revetments appear to have a sufficient median armor stone diameter for the 10-year, 20-year, 50-year, and 100-year return period events. However, if future revetment improvements intend to change the structure crest elevation, toe elevation, or slope, it is recommended that the stone size be re-evaluated.

7.2 APPENDIX B – FDOT COMPLEX PIER LOCAL SCOUR CALCULATOR (VERSION 6.2) – PARAMETERS FOR THE 50-, 100-, AND 500-YEAR LOCAL SCOUR

50-YEAR

This program was developed for use with cohesionless sediment.

Calculate Scour

Effective Diameter, D* (ft)	6.4
Local Scour, ys (ft)	6.7

Flow and Sediment

D_{50} (mm)	0.1
Angle of Attack (°)	30
y_o (ft)	14.1
V (ft/s)	2.5

Column Data

b_{col} (ft)	10
L_{col} (ft)	6
H_{col} (ft)	-3
Shape	Rectangular

☐ No Column

Pile Cap Data

b_{pc} (ft)	22
L_{pc} (ft)	16.5
T (ft)	7
H_{pc} (ft)	-10
Shape	Rectangular

☐ No Pile Cap

Pile Group Data

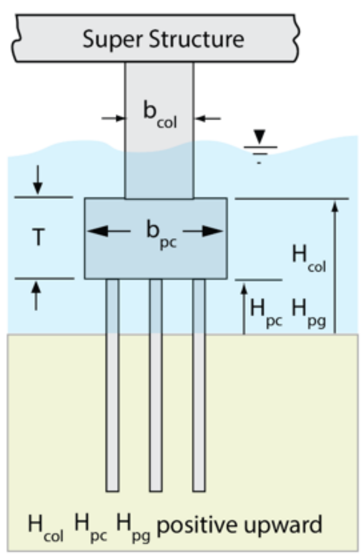
n	2
m	3
b (ft)	2
s_n (ft)	18
s_m (ft)	6
H_{pg} (ft)	-10
Shape	Rectangular

☐ No Pile Group

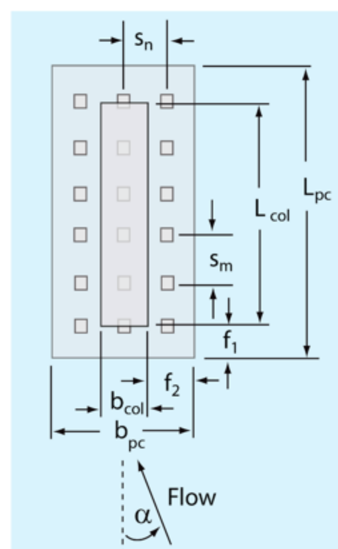
Project Information for Reporting

Bridge Number	123
Route	34
Waterway	345
Pier Number	324
Return Period	6
Created By	fgb
Date	01/01/1000

Create Report



Front View



Top View

100-YEAR:

This program was developed for use with cohesionless sediment.

Calculate Scour

Effective Diameter, D* (ft)	6.6
Local Scour, ys (ft)	6.2

Flow and Sediment

D_{50} (mm)	0.1
Angle of Attack (°)	30
y_o (ft)	15.2
V (ft/s)	1.6

Column Data

b_{col} (ft)	10
L_{col} (ft)	6
H_{col} (ft)	-3
Shape	Rectangular

☐ No Column

Pile Cap Data

b_{pc} (ft)	22
L_{pc} (ft)	16.5
T (ft)	7
H_{pc} (ft)	-10
Shape	Rectangular

☐ No Pile Cap

Pile Group Data

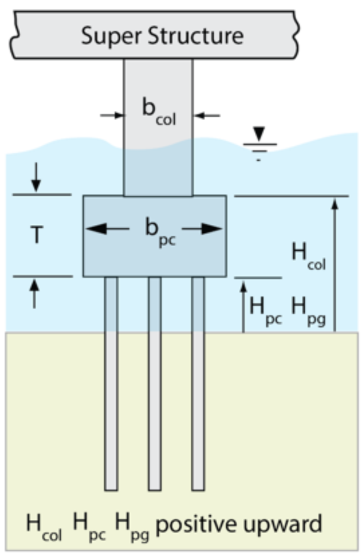
n	2
m	3
b (ft)	2
s_n (ft)	18
s_m (ft)	6
H_{pg} (ft)	-10
Shape	Rectangular

☐ No Pile Group

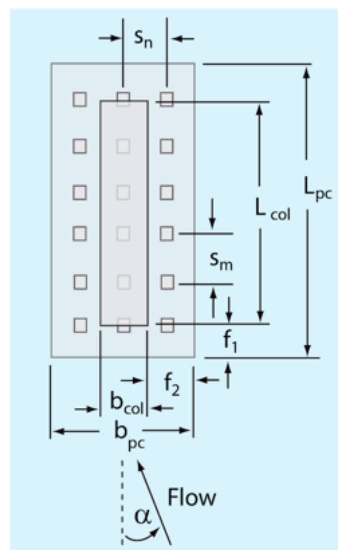
Project Information for Reporting

Bridge Number	123
Route	34
Waterway	345
Pier Number	324
Return Period	6
Created By	fgb
Date	01/01/1000

Create Report



Front View



Top View

500-YEAR:

This program was developed for use with cohesionless sediment.

Calculate Scour	
Effective Diameter, D* (ft)	6.6
Local Scour, ys (ft)	6.2

Flow and Sediment

D_{50} (mm)	0.1
Angle of Attack (°)	30
y_o (ft)	17
V (ft/s)	1.5

Column Data

b_{col} (ft)	10
L_{col} (ft)	6
H_{col} (ft)	-3
Shape	Rectangular

☐ No Column

Pile Cap Data

b_{pc} (ft)	22
L_{pc} (ft)	16.5
T (ft)	7
H_{pc} (ft)	-10
Shape	Rectangular

☐ No Pile Cap

Pile Group Data

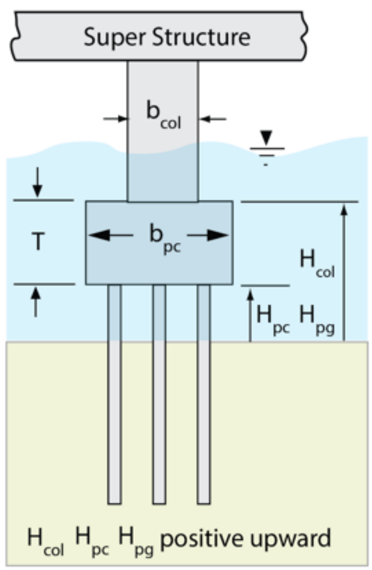
n	2
m	3
b (ft)	2
s_n (ft)	18
s_m (ft)	6
H_{pg} (ft)	-10
Shape	Rectangular

☐ No Pile Group

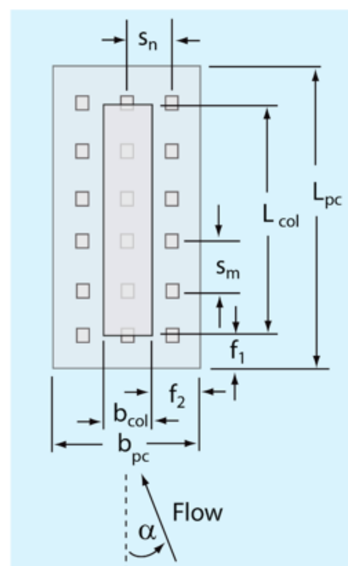
Project Information for Reporting

Bridge Number	123
Route	34
Waterway	345
Pier Number	324
Return Period	6
Created By	fgb
Date	01/01/1000

Create Report



Front View



Top View

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Bay Harbor Islands, FL 33154
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