ORDINANCE NO. 2021-006

AN ORDINANCE OF THE MAYOR AND COMMISSION OF NORTH BAY VILLAGE, FLORIDA, AMENDING THE VILLAGE UNIFIED LAND DEVELOPMENT CODE OF ORDINANCES BY CREATING SECTION 8.16, ENTITLED, "SEA WALL STANDARDS," IN DIVISION 4, "SUPPLEMENTAL DEVELOPMENT STANDARDS," CHAPTER 8, "ZONING"; ESTABLISHING SEA WALL DESIGN STANDARDS THROUGHOUT THE VILLAGE; PROVIDING FOR CONFLICTS; PROVIDING FOR SEVERABILITY; PROVIDING FOR INCORPORATION; PROVIDING FOR IMPLEMENTATION; AND PROVIDING FOR AN EFFECTVE DATE

WHEREAS, North Bay Village (the "Village") strives to ensure that its infrastructure is resilient against flooding, significant weather events, sea level rise, and other consequences of climate change, so as to protect all property within the Village and the future quality of life of Village residents; and

WHEREAS, the Village recognized that it had very limited policies in its Unified Land Development Code regarding the construction of a critical piece of Village infrastructure—seawalls; and

WHEREAS, in 2019, the National Oceanic and Atmospheric Administration updated its sea level rise predictions, calling on sea level to rise by seventeen (17) inches by 2040 and as much as forty (40) inches; and

WHEREAS, as part of its recent master planning study, "NBV 100," the Village identified resilient design and sustainability as a community wide approach, calling for the adaptability of seawalls to better protect the Village's shoreline; and

WHEREAS, in 2020, the Village commission consultants to prepare seawall design criteria so as to make uniform policies that, overtime, would result in improved and hardened sea walls throughout the Village; and

WHEREAS, the Commission desires to incorporate those sea wall standards into the Unified Land Development Code; and

WHEREAS, the Mayor and Commission believe that this Ordinance is in the best interest of the Village, as it advances the health, safety, and wellness of the community

NOW, THEREFORE, IT IS HEREBY ORDAINED BY MAYOR AND COMMISSION OF NORTH BAY VILLAGE, FLORIDA, AS FOLLOWS:

Section 1. Recitals Adopted. The foregoing recitals are confirmed, adopted, and incorporated herein and made a part hereof by this reference.

Section 2. Village Code Amended. The Village Code of Ordinances is hereby amended as follows:

Chapter 8 - ZONING

* * *

DIVISION 4 – SUPPLEMENTAL DEVELOPMENT STANDARDS

* * *

Section 8.16 – Sea Wall Standards

In addition to all other requirements of this Code, all properties directly abutting North Bay Village's shoreline, upon, and as part of, their development or substantial redevelopment of, shall be required to design, construct, and otherwise conform their respective sea walls to the design criteria and standards established in the Seawall Design Criteria Project Memorandum, dated October 19, 2020, prepared by Moffatt & Nichol for the Village, as may be updated from time to time, which is incorporated into the Code as Appendix "A" and made a part hereof by reference.

* * *

Section 3. Conflicts. All Sections or parts of Sections of the Code of Ordinances, all ordinances or parts of ordinances, and all Resolutions, or parts of Resolutions, in conflict with this Ordinance are repealed to the extent of such conflict.

<u>Section 4.</u> **<u>Severability.</u>** That the provisions of this Ordinance are declared to be severable and if any section, sentence, clause or phrase of this Ordinance shall for any reason be held to be invalid or unconstitutional, such decision shall not affect the validity of the remaining sections, sentences, clauses, and phrases of this Ordinance but they shall remain in effect, it being the legislative intent that this Ordinance shall stand notwithstanding the invalidity of any part.

<u>Section 5.</u> **<u>Inclusion in Code.</u>** The provisions of this Ordinance shall become and be made a part of the Village Code, that the sections of this Ordinance may be renumbered or relettered to accomplish such intentions, and that the word Ordinance shall be changed to Section or other appropriate word

<u>Section 6.</u> **<u>Implementation.</u>** The Village Manager, Village Attorney, and Village Clerk are hereby authorized to take such further action as may be needed to implement the purpose and provisions of this Ordinance

<u>Section 7.</u> **<u>Effective Date.</u>** That this Ordinance shall become effective immediately upon adoption on second reading.

The foregoing Ordinance was offered by Commissioner Strout who moved its adoption on final reading. This motion was seconded by Commissioner Dr. Chervony and upon being put to a vote, the vote was as follows:

Mayor Brent Latham	YES
Vice Mayor Marvin Wilmoth	YES
Commissioner Richard Chervony	YES
Commissioner Rachel Streitfeld	YES
Commissioner Julianna Strout	YES

PASSED on first reading on this 8th of December, 2020.

PASSED AND ENACTED on second reading on this 9th day of March, 2021.

____
Brent Latham, Mayor

ATTEST:

____
Elora Riera, CMC
Village Clerk

APPROVED AS TO LEGAL SUFFICIENCY:

____
Weiss Serota Helfman Cole & Bierman, PL
Village Attorney

moffatt & nichol
2937 SW 27th Avenue, Suite 101A
Miami, Florida 33133

(305) 230-1924
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EAC Consulting, Inc.
eacconsult.com

MEMORANDUM

To:	**Ralph Rosado**
From:	**Abbie Wilson, PE**
Prepared By:	**Abbie Wilson (M&N), Evelyn Rodriguez (EAC)**
Date:	**October 19, 2020**
Subject:	**Seawall Design Criteria Package - Introduction Memo**
M&N Job No.:	**10907**

In an effort to preserve public and private properties along the North Bay Village coastline against sea level rise and wave action, EAC was retained by the Village to develop a minimum design criteria for new seawalls to be designed and constructed within the Village to facilitate overall resiliency and coastal defense that could be adopted through code development, otherwise known as the Seawall Design Criteria Package (DCP).

North Bay Village currently has very limited code regarding seawall construction within its limits. Three separate projects for the replacement or repair of seawalls within commercial properties were permitted through the Village in 2017 and another in 2018, all of which had different seawall cap elevations. The cap elevations ranged from 2.64 feet (ft) North American Vertical Datum (NAVD) [4.20 ft National Geodetic Vertical Datum (NGVD)] to 6.73 ft NAVD (8.29 ft NGVD) highlighting the need for an updated and consistent requirement within the Village.

Over the past several years, several cities and counties have adopted updated code requirements for seawalls. Existing code was reviewed from surrounding communities within Southeast Florida. Those communities included:

- City of Miami Beach,
- City of Miami (Miami21),
- City of Ft. Lauderdale,
- Town of Bay Harbor Islands, and
- Broward County.

The two most stringent ordinances are for the City of Miami Beach and the City of Miami. The City of Miami Beach has a minimum cap elevation for public properties set at 5.70 ft NAVD (7.26 ft NGVD), while the proposed ordinance for the City of Miami, not yet adopted, will set the minimum cap elevation at 6.0 ft NAVD (7.55 ft NGVD).

In 2019, the sea level rise estimates were updated with the latest predictions from National Oceanic and Atmospheric Administration (NOAA). The NOAA Intermediate High is typically used in planning for future sea level rise. Based on the latest trend, 17 inches of sea level rise by the year 2040 is expected, and 40 inches of sea level rise is anticipated by 2070.

North Bay Village recently completed a master planning study highlighting resilient design and sustainability as a community wide approach titled NBV100 by DPZ CoDESIGN. NBV100 calls for the adaptability of seawalls and for the use of innovative design, such as Glass Fiber Polymer Reinforced (GFPR) Concrete Seawalls and living shorelines. However, any of the following seawall design options are applicable to the shoreline of North Bay Village including concrete pile and panel walls, University of Miami's Glass Fiber Polymer Reinforced Concrete Wall, and steel or composite sheet pile walls, as well as living shorelines.

After review of the available data on past projects, the required cap elevations for adjacent communities, and predicted sea level rise, a minimum cap elevation of 5.94 ft NAVD (7.5 ft NGVD) was recommended. The design of the future seawalls must also account for an additional 2.5 ft of cap elevation for the seawalls to be raised in the future without the need for redesign. The seawalls will also need to account for not only conventional "bulkhead" design requirements but will also need to account for coastal engineering loads. These loads include wave impact, storm surge, wave overtopping, differential water levels, and seepage. The seawalls will facilitate resiliency; however, groundwater is also a concern which is not addressed by seawalls and will require further evaluation by the Village. Seawall projects will additionally need to be permitted by the U.S. Army Corps of Engineers, Florida Department of Environmental Protection, Miami-Dade Department of Regulatory and Economic Resources, and the North Bay Village Building Department.

The walls are expected to range in cost from $1,800 to $3,800 per linear foot based on the geotechnical information outlined in this report. The table below provides a breakdown of the wall options outlined within the Seawall DCP and the associated approximate cost. Additionally, the Village will need to consider the requirement for existing public and private property owners to upgrade their existing seawalls to these new requirements within an established period of time.

Approximate cost range for various wall types*	
Wall Type	Approx. Cost Range
Concrete Pile and Panel	$2,200 to $3,100 per LF
University of Miami GFPR Concrete Seawall	10% > than Standard Concrete Walls
Steel or Composite Sheet Pile	$1,800 to $ $3,800 per LF
Living Shoreline	$220 to $290 per CY of Riprap**
Raise Existing Seawall	Unknown***

*All wall options must have minimum cap elevations of 5.94 NAVD (7.5 NGVD) with the ability to increase the cap by an additional 2.5 ft without the need for redesign.
**Volume of riprap may vary greatly. Cost does not include permitting expense.
*** Cost to raise a seawall will vary significantly based on existing wall section.

The attached Seawall Design Criteria Project Program Memorandum outlines the background information used to create a seawall design criteria package that, if codifed, provides resiliency by adapting to rising sea levels, reducing storm surge risk, and works within the physical constraints of the Village. The Village exists entirely on reclaimed land within Biscayne Bay; therefore, shoreline stabilization is considered critical infrastructure. The information contained within the Seawall DCP does not constitute an engineering design. Seawalls will need to be designed and permitted by a licensed engineer.

Seawall Design Criteria
Project Program Memorandum
North Bay Village, Miami-Dade County, Florida

Presented to:

North Bay Village
1666 Kennedy Causeway, 3rd Floor
North Bay Village, FL 33141

EAC Consulting, Inc.

5959 Blue Lagoon Drive
Suite 410
Miami, FL 33126

Prepared by:

moffatt & nichol

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Document Verification

Client	North Bay Village
Project name	Seawall Design Criteria Package
Document title	Project Program Memorandum
Document sub-title	N/A
Status	DRAFT
Date	07/01/2020
Project number	10907
File reference	

Revision	Description	Issued by	Date	Checked
00	DRAFT	AEW	4/16/2020	LC
01	DRAFT - SRTF	AEW	6/25/2020	
02	DRAFT – July Commission	AEW	7/1/2020	LC
03	FINAL – September Commission	AEW	8/21/2020	LC
04	FINAL – Planning and Zoning	AEW	10/19/2020	LC

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Executive Summary

In an effort to preserve public and private properties along the North Bay Village coastline against sea level rise and wave action, EAC was retained by the Village to develop a minimum design criteria for new seawalls to be designed and constructed within the Village to facilitate overall resiliency and coastal defense through code development.

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The Southeast Florida Regional Climate Change Compact is the leading source of information on Sea Level Rise in Southeast Florida. In 2019, the sea level rise estimates were updated with the latest predictions from National Oceanic and Atmospheric Administration (NOAA). The NOAA Intermediate High is typically used in planning for future sea level rise. Based on the latest trend, 17 inches of sea level rise by the year 2040 is expected, and 40 inches of sea level rise is anticipated by 2070.

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The following outlines the recommended criteria for inclusion in the seawall code development.

- Minimum Cap Elevation
  - 5.94 ft NAVD (7.50 ft NGVD) with ability to raise cap additional 2.5 ft in future
- Coastal Engineering Considerations
  - Wave impact
  - Storm Surge
  - Hydrostatic loads due to tidal prism lag
  - Toe scour
  - Overtopping
  - Seepage
- Minimum Design Service Life
  - 25 Years
- Allowable Seawall Configurations
  - Concrete Pile and Panel Walls
  - University of Miami's Glass Fiber Polymer Reinforced Concrete Wall
  - Sheet Pile Wall
    - Steel
    - Composite
  - Living Shorelines.

The walls are expected to range in cost from $1,800 to $3,800 per linear foot based on the geotechnical information outlined in this report. The table below provides a breakdown of the wall options outlined herein and the associated approximate cost. Additionally, the Village will need to consider the requirement for existing public and private property owners to upgrade their existing seawalls to these new requirements within an established period of time.

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This Project Program Memorandum outlines the background information used to create a seawall design criteria code that provides resiliency by adapting to rising sea levels, reducing storm surge risk, and works within the physical constraints of the Village. The Village exists entirely on reclaimed land within Biscayne Bay; therefore, shoreline stabilization is considered critical infrastructure. The information contained herein does not constitute an engineering design. Seawalls will need to be designed and permitted by a licensed engineer.

1. Introduction

North Bay Village, an island community of about 8,000 residents along NE 79th Street/Kennedy Causeway, is impacted by the effects of sea level rise with increased storm surge inundation and “sunny day” flooding, flooding cause by “king tide” events backing up into the Village’s stormwater collection system. The highest king tide on record occurred on October 2017 at an elevation of 2.27 ft NAVD (3.83 ft NGVD).

North Bay Village is made up of three reclaimed islands that sit halfway between the cities of Miami and Miami Beach in the middle of the Biscayne Bay Aquatic Preserve as shown in Figure 1-1. In 1940, dredging, bulkheading, and filling created North Bay Island. During the mid-1940s, additional dredging, bulkheading, and filling created the additional two islands to the north and east, Harbor Island and Treasure Island. The islands are a mix of single-family, multi-family, and commercial properties. Over the years very little has been done to improve the condition of the seawalls since their original construction in the mid-1900s, except for the replacement at a few select properties. The purpose of this memorandum is to aid the Village in establishing minimum criteria for upgrading the seawalls that surround the entire community that can be implemented as a code revision. The more stringent criteria will facilitate overall resiliency by reducing the risk from storm surge and increasing coastal defense throughout the Village. The Village’s shoreline is considered critical infrastructure as shoreline stabilization is essential to the protection of private and public property.

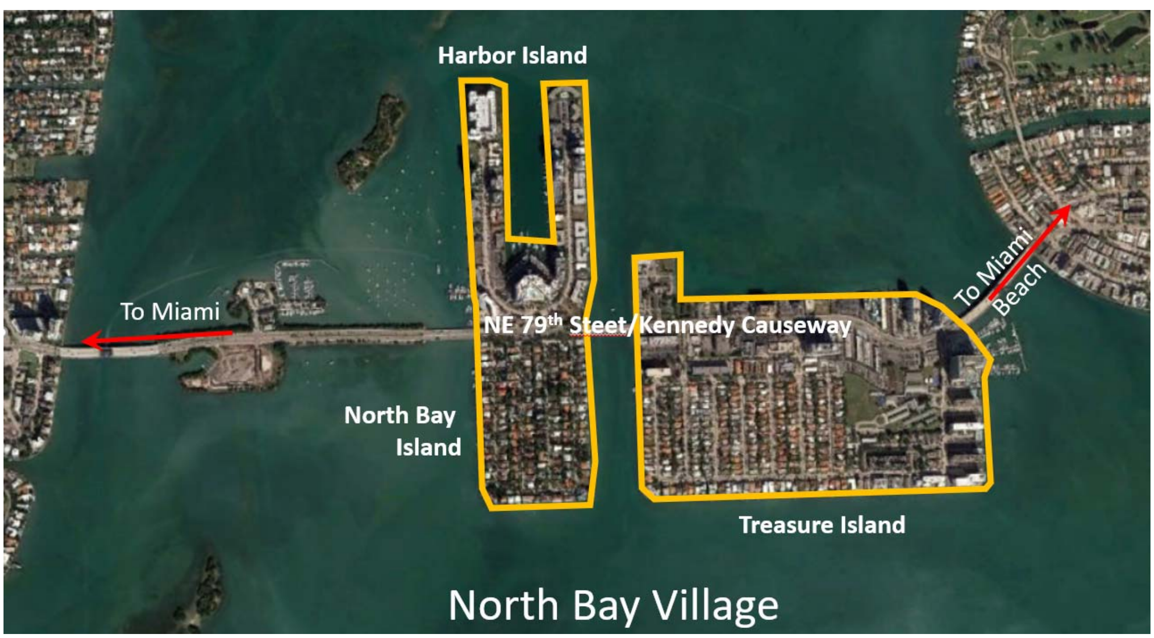

Figure 1-1: Location Map (Source: Google Earth)

2. Existing Data Collection

Data were collected from previous seawall projects constructed within North Bay Village, previous resiliency studies that include the project area, and seawall guidelines for the surrounding communities to provide guidance on setting an acceptable minimum seawall elevation.

2.1. Existing Seawall Guidelines

The Village does not currently have an ordinance in place mandating seawall improvements, required heights, or construction standards. The existing guidance, found in Division 5 § 9.11 and 9.12, describes the maximum horizontal offsets and the requirement for being in compliance with the Miami-Dade County Shoreline Development Review (Ordinance 85-14).

Figure 2-1 shows the Future Land Uses from the 2026 Comprehensive Plan highlighting the distribution of public and private properties. According to the land use, about 30 percent of the seawalls are privately owned by single-family properties.

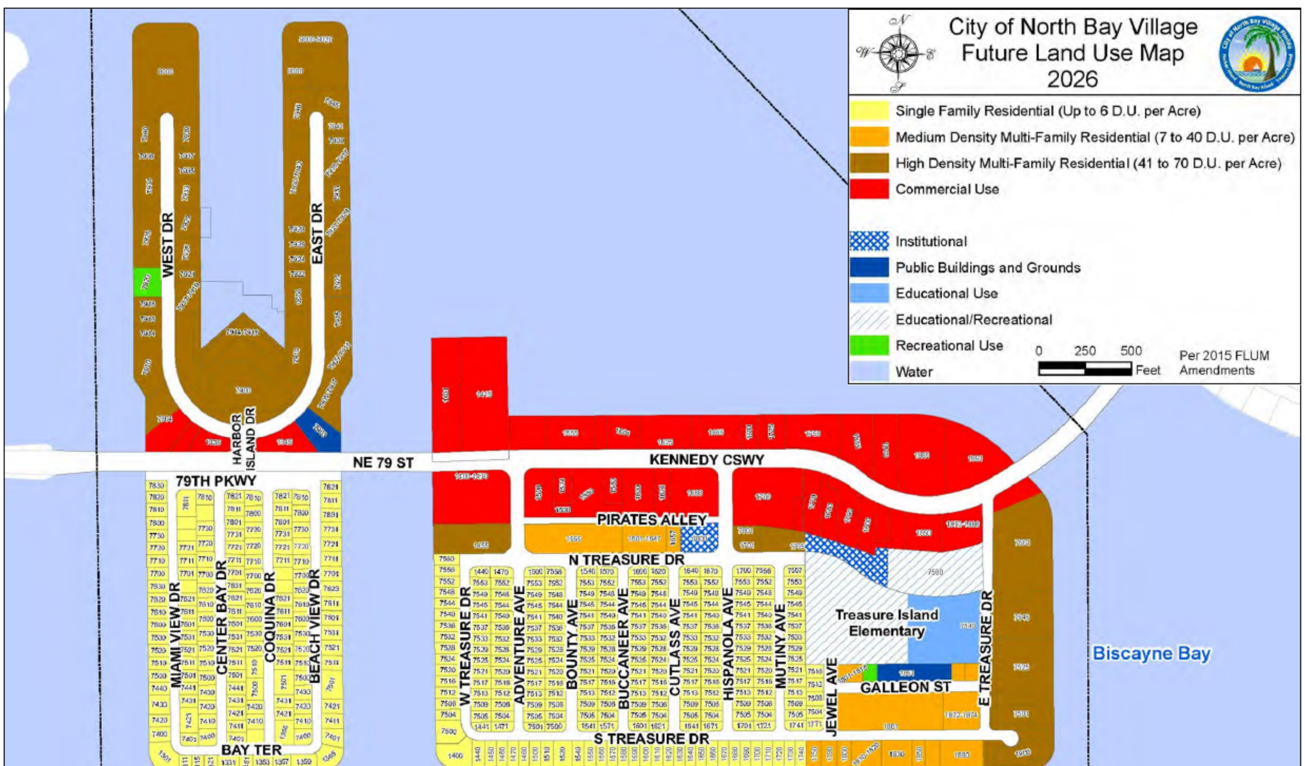

Figure 2-1: North Bay Village Land Use (North Bay Village Comprehensive Plan, July 2018)

2.2. Recent North Bay Village Seawall Projects

In 2017, three separate seawall projects were permitted and constructed within the Village, all along NE 79th Street, all with varying top of wall elevations.

- Benihana Restaurant
  - Reconstruction seaward of existing wall
  - Steel sheet pile wall with king and batter piles
  - Concrete Cap
  - Top of Wall Elevation = 7.50 ft NGVD
- Shucker's Restaurant
  - Extension and raising of existing seawall combined with reconstruction of existing wall
  - Concrete pile and panel with king and batter piles
  - Existing walkway over water remained
  - Existing seawall raised to elevation 5.04 ft NGVD with cap addition
  - Top of newly reconstructed seawall built to elevation 6.60 ft NGVD
- Vacant Lot at 1725 NE 79th Street
  - Repair of existing seawall
  - Installed concrete batter piles and raised concrete cap
  - Existing seawall raised to elevation 4.20 ft NGVD

With permitting that began in 2018 and construction finishing in May 2020, the most recent seawall constructed in North Bay Village was on the southeast corner of Treasure Island at Treasures on the Bay. This project reconstructed the existing seawall utilizing concrete pile and panel with king and batter piles with a cap elevation of 8.29 ft NGVD. In order to provide access to the floating dock installed with this project, the cap was lowered to an elevation of 6.29 ft NGVD for five feet. Permitted plans for the seawall at the Benihana restaurant and Treasures on the Bay are included in Appendix A.

2.3. Previous Resiliency Studies

The Southeast Florida Regional Climate Change Compact is a decade-old partnership between Broward, Miami-Dade, Monroe, and Palm Beach counties, to work collaboratively and build climate resilience and sustainability within their own communities and across the Southeast Florida region. The Climate Compact is looked to as the leading source of information on sea level rise. In 2019, the sea level rise estimates were updated with the latest predictions (Figure 2-2) from National Oceanic and Atmospheric Administration (NOAA). The NOAA Intermediate High is typically used in planning for future sea level rise. Based on the latest trend, 17 inches of sea level rise by the year 2040 is expected, and 3 ft of sea level rise by about 2065.

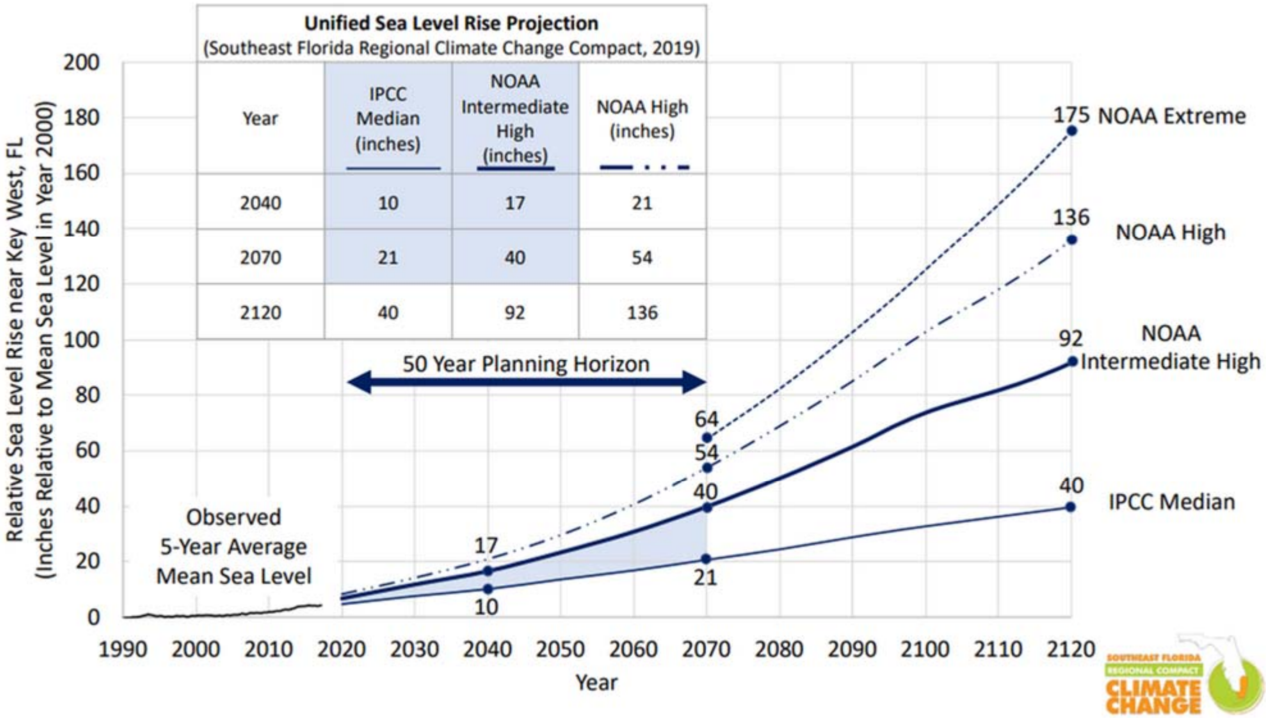

Figure 2-2: Southeast Florida Regional Climate Change Compact Unified Sea Level Rise Projections (2019)

Flooding associated with rising seas is already being experienced within the Village in the form of "sunny day flooding" and "king tides". "Sunny day flooding" occurs as a result of seawater backing up into the Village's stormwater infrastructure, rising out of the inlets, without an associated rain event. "King tides" are the highest astronomical tides that occur a few times a year, on average. As the sea level rises, "sunny day flooding" will happen more frequently and "king tides" will have an even greater impact on the community.

In order to predict the effects the rise in sea levels will have on communities and infrastructure in Southeast Florida, multiple agencies have developed models to visualize the impacts. These visualizations help communities determine vulnerable areas in order to plan for a resilient future. Eyes on the Rise is a project within the Sea Level Rise Tool Box, created at Florida International University's Sea Level Rise: South Florida initiative. The goal of the Sea Level Rise Tool Box is to inform citizens of South Florida about the potential impact of sea level rise in their neighborhoods. When North Bay Village is viewed through the Tool Box, varying flood impacts are shown as a result of sea level rise. Figure 2.3 demonstrates 3 ft of sea level rise within the North Bay Village Islands. This figure highlights the vulnerable areas of the islands.

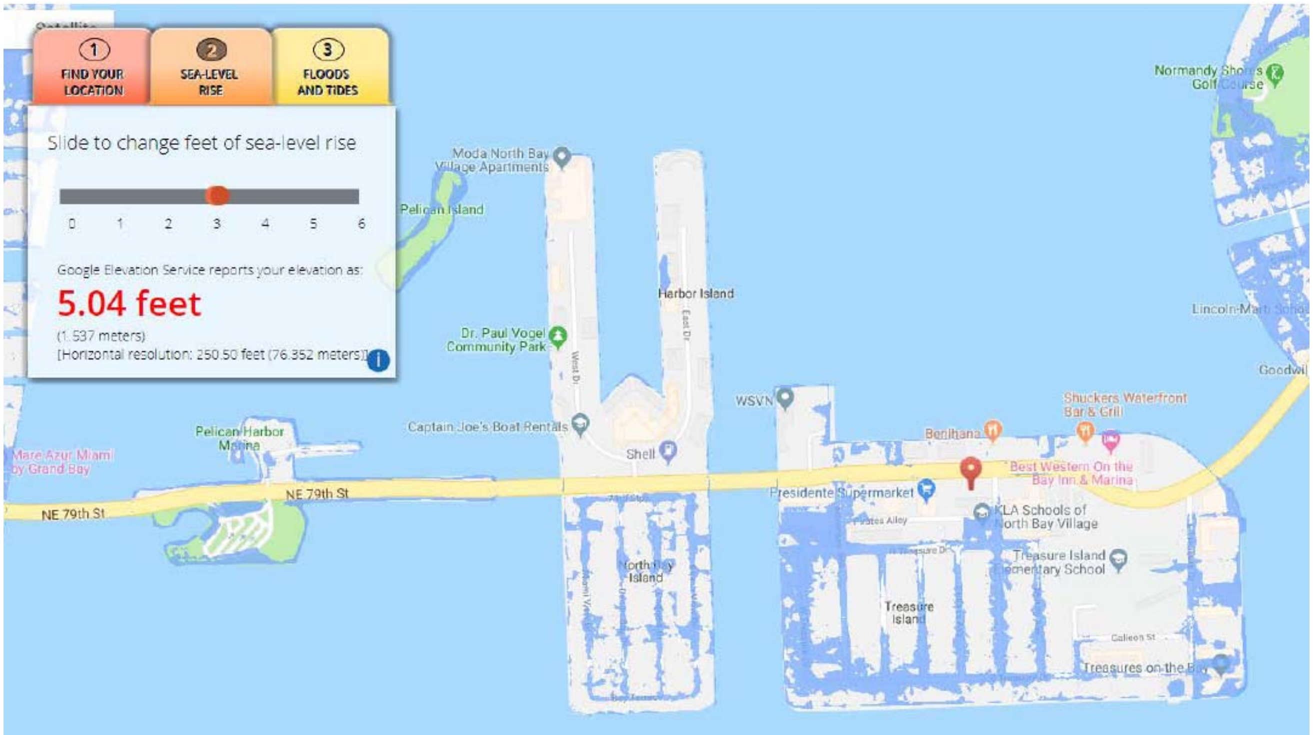

Figure 2-3: Three Feet of Sea Level Rise in North Bay Village as Simulated by FIU Eye on the Rise Project

The sea level rise viewer produced by NOAA produces very similar results. Figure 2.4 shows North Bay Village with 3 ft of sea level rise as calculated by NOAA.

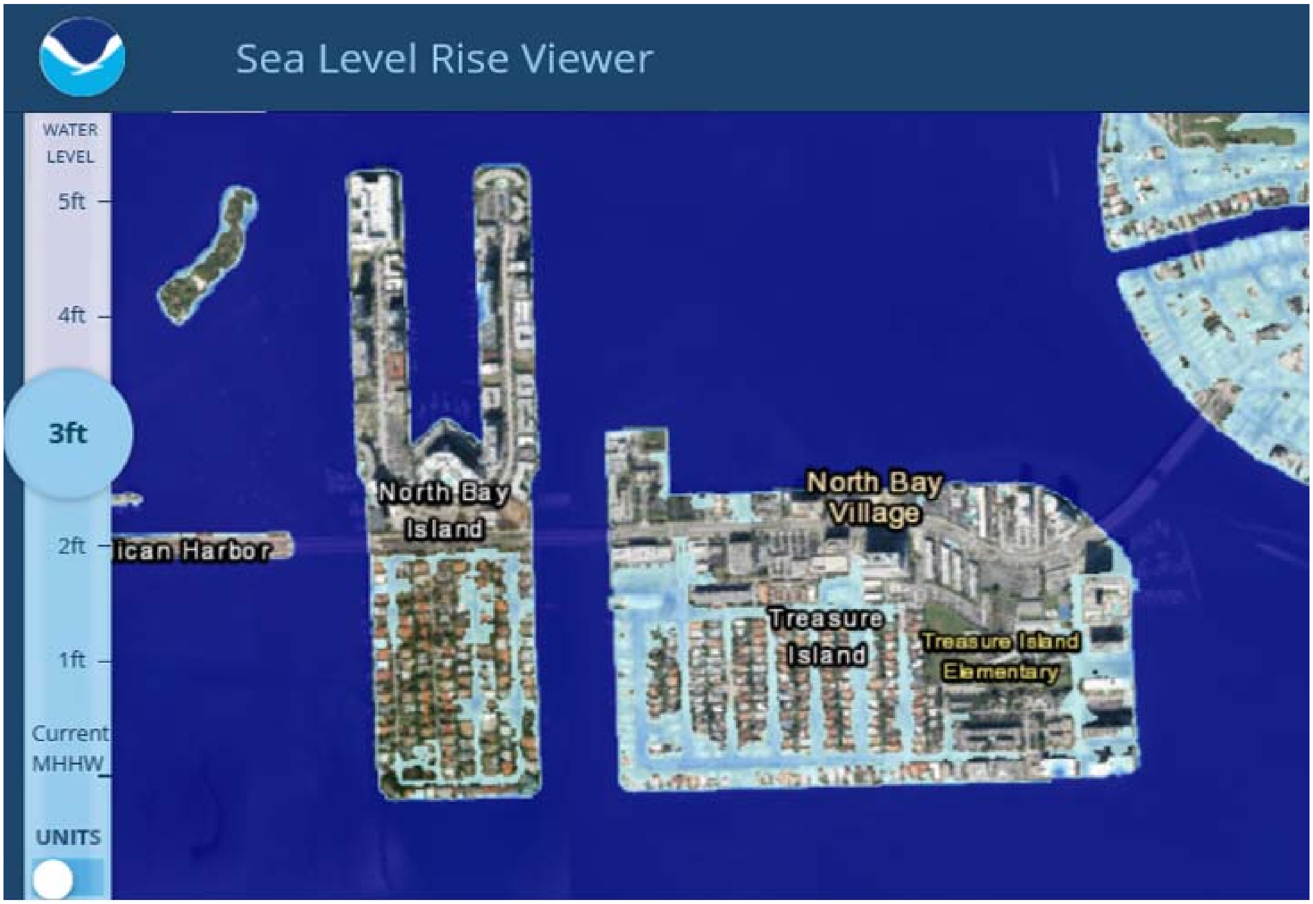

Figure 2-4: NOAA Sea Level Rise Viewer Simulation 3 ft of Sea Level Rise in North Bay Village

Miami-Dade County created a GIS Map showing building impacts associated with various sea level rise scenarios utilizing the flooding data from the NOAA model. Figure 2.5 shows the buildings at moderate (orange) and high (red) risk of being impacted by 3 ft of sea level rise.

Figure 2-5: Miami-Dade County Building Impact Viewer with 3 ft of Sea Level Rise

The flooding locations, shown in the aforementioned models, agree with the low areas of the islands based on the 2015 LiDAR data. Figure 2-6 shows the 2015 Miami-Dade County 5 ft LiDAR covering the area of North Bay Village, highlighting the areas with the lowest existing seawalls, contributing to the predicted flooding.

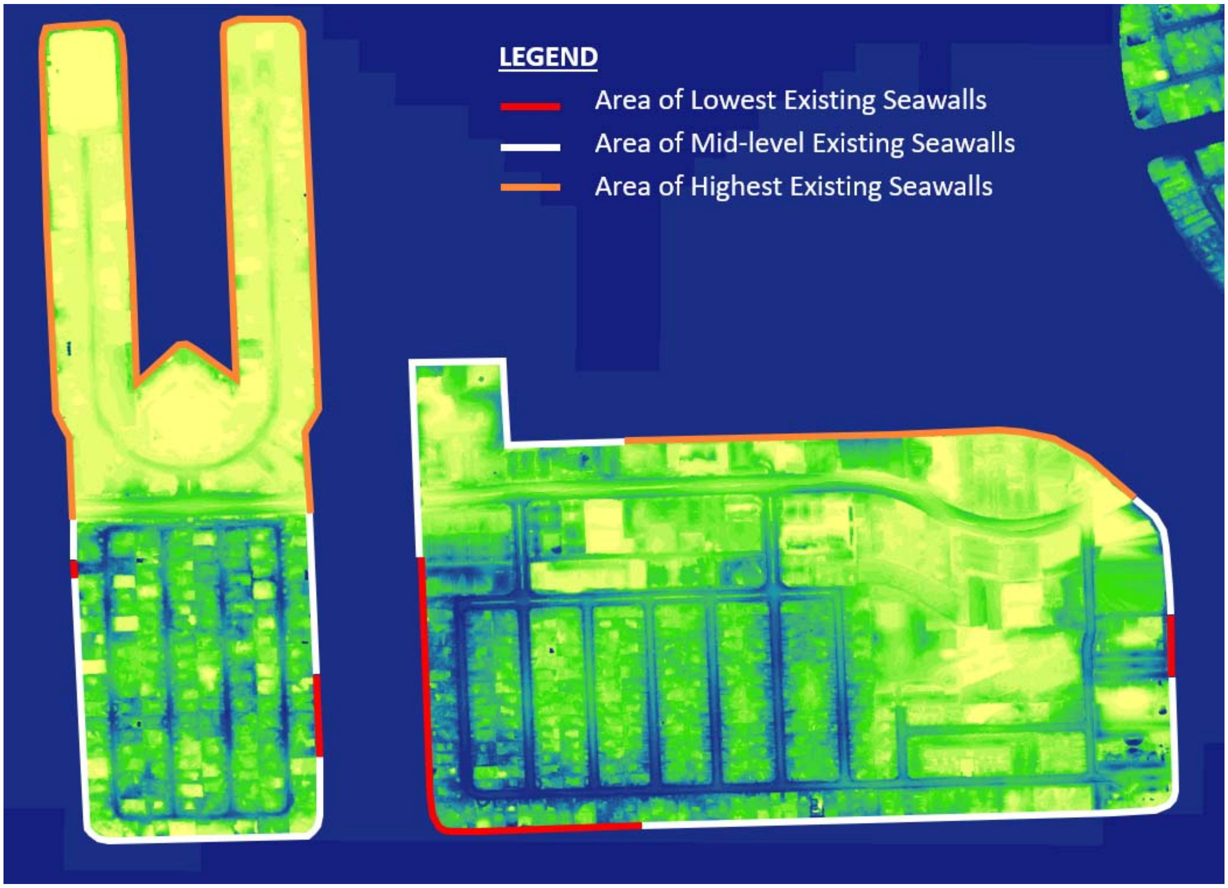

Figure 2-6: Miami-Dade County 5 ft LiDAR with Seawalls Heights

2.4. Previous North Bay Village Planning Studies

In 2015, a conceptual plan was created for a boardwalk to be installed along the full length of the north side of Harbor Island, very similar in location to the proposed Island Walk described in the NBV100 Planning Study, except it was to have been installed over water. This design did not include the raising of the seawalls.

More recently, NBV100, the master planning study charting the path for a more livable, prosperous, and resilient North Bay Village, performed by DPZ CoDESIGN was completed in March 2020. This master plan outlines the necessity for a standard seawall design criteria as seawalls are the main infrastructure protecting the islands against storm surge and sea level rise. NBV100 recommends the use of innovative and adaptable designs, such as the use of fiber-reinforced concrete and the ability to raise seawall elevations as much as 2.5 ft in the future without the need for rebuilding. The study also recommends raising the minimum elevation of the seawall.

2.5. Guidelines from Other Municipalities

South Florida is at the forefront of combatting sea level rise and Miami-Dade and Broward Counties are no exception. Broward County, along with a number of municipalities located within Southeast Florida, have established seawall design criteria and resiliency guidelines that address the latest sea level rise concerns facing their communities. Table 2-1 presents the latest seawall height guidelines in reference to their written datum.

Table 2-1: Minimum Seawall Elevation Criteria by Agency

Date Set	Agency	Height Requirement**
Nov. 2010	City of Miami - Miami 21	5.0' NGVD N. of Rickenbacker Cswy
		6.0' NGVD S. of Rickenbacker Cswy
May 2016	City of Miami Beach	5.7' NAVD (7.26' NGVD) (Public Walls)
		4.0' NAVD (Private Walls Only)
Dec. 2016	City of Ft. Lauderdale	3.9' NAVD (minimum)
		5.0' NAVD (recommended)
		Max wall height set at FEMA BFE
Feb. 2018	Town of Bay Harbor Islands*	5.50' above MSL
		6.0' above MSL if more frequent and turbulent wave action
Feb. 2020	Broward County	4.0' NAVD (by 2035)
		5.0' NAVD (by 2050)

* No datum is provided in Ordinance other than MSL

** Height requirements are shown in datums referenced in their respective code.

The City of Miami has a proposed ordinance under development to set the required seawall height to 6 ft NAVD (7.55 ft NGVD) for the entire city with the ability to increase the cap by an additional 2 ft. The ordinance for Broward County requires all tidally influenced municipalities within its limits to adopt the revised code within 24 months of its issuance. Within unincorporated Broward County, all seawalls that new or have substantial improvements have to immediately comply with the revised code, while damaged or structurally deficient seawalls will have a period of 365 days from notification from the County to repair or replace the seawall and bring it into compliance with the new code. Miami-Dade County does not have a minimum seawall elevation set but requires all county infrastructure to account for sea level rise in the design and construction, as outlined in Ordinance 14-79.

For the purpose of this memorandum, the conversion between NGVD and NAVD is +1.56 ft NGVD = 0.00 ft NAVD.

3. Criteria Development

The design of a resilient seawall is a combination of many factors. Outlined in this section are the prominent factors that will affect the design of seawalls within North Bay Village.

3.1. Cap Elevation

The proposed ordinance for Miami21 proposes to raise the minimum seawall elevation to 6.0 ft NAVD (7.55 ft NGVD). The City of Miami Beach utilizes a minimum cap elevation of 5.70 ft NAVD (7.26 ft NGVD) for public properties. The new seawall installed along North Bay Village's Benihana's waterfront has a cap elevation of 5.94 ft NAVD (7.50 ft NGVD). After reviewing the predicted sea level rise and the minimum cap elevations proposed by various governing bodies, the suggested minimum seawall elevation is 5.94 ft NAVD (7.5 ft NGVD). To be in agreement with the NBV100 Study, the structural design of the seawall must also account for the ability to raise the cap in the future by an additional 2.5 feet to an elevation of 8.44 ft NAVD (10.0 ft NGVD) in the future without redesign of the seawall. This adaptability is also appropriate for planning purposes due to uncertainties with sea level rise predictions in the long-term. Figure 3-1 shows both the existing and predicted future (Year 2070) water levels in relation to the proposed seawall cap elevation of 5.94 ft NAVD.

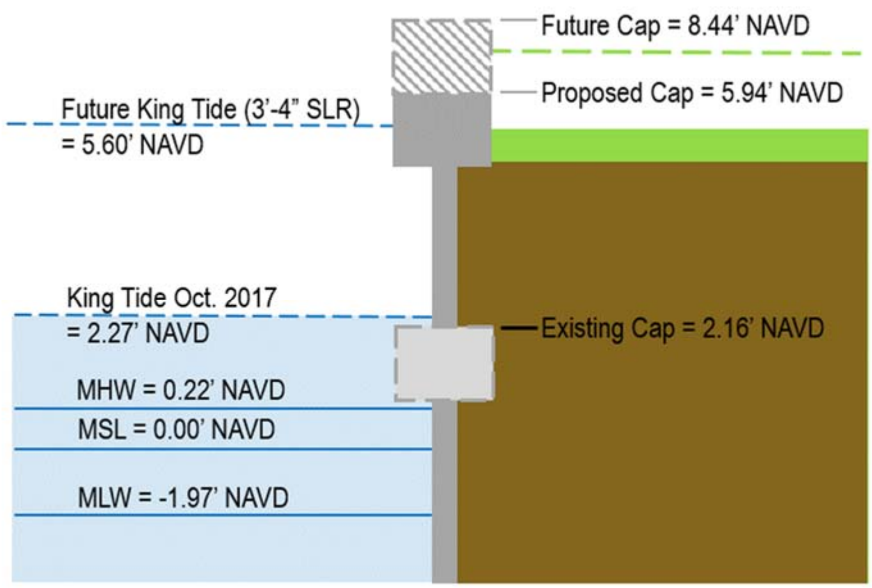

Figure 3-1: Proposed Cap Elevation in relation to Existing and Future Water Levels

3.2. Service Life

Seawalls in North Bay Village should be designed and constructed to provide a service life of at least 25 years. Twenty-five years is the design service life specified by the U.S. Navy Waterfront Criteria. With the use of standard marine materials in a typical environment, structural components may begin to deteriorate after 25 years.

Materials should be specified for the marine environment and construction practices monitored for the marine structures to provide the longest service life possible with minimal maintenance. South Florida exhibits high corrosion rates on materials such as steel with the tropical conditions and salinity of the surrounding Biscayne Bay. Advanced materials and systems are available, including systems developed by the University of Miami referenced in Section 4.2, that can provide a service life of 50+ years.

3.3. Geotechnical Considerations

In March 2020, GCES Engineering Services, LLC. (GCES) performed three geotechnical borings: one on Harbor Island and two on Treasure Island. The borings are consistent with the site history and show fill material for the top 8 to 12 feet. The top of the limestone was encountered in all three boring locations at a depth of 17 to 20 ft, while the groundwater table was found at a depth of 4.5 ft to 7 ft below existing grade. The geotechnical conditions documented in the GCES report, found in Appendix B, provide general guidance relative to subsurface conditions in the Village and this information can be supplemented by site-specific evaluation for engineering design.

3.4. Groundwater Levels

According to the Miami-Dade Average October Groundwater Map WC 2.2, the wet season groundwater in North Bay Village is approximately at elevation 0.44 ft NAVD (2 ft NGVD). The groundwater fluctuates slightly with the tide levels, but at a much smaller amplitude. In 2015, the USGS, in conjunction with Miami-Dade Water and Sewer, performed an extensive study on the effect sea level rise and well drawdown on the groundwater elevations. The study found for coastal areas of Miami-Dade County, of which North Bay Village is included, the groundwater rise had a one-to-one ratio with sea level rise. This means for every foot of sea level rise, the groundwater will rise one foot as well. The construction of seawalls alone, no matter the height, will not protect the island from rising seas given the connectivity with the groundwater and storm sewer system. Further evaluation of the effects of groundwater on resiliency in the Village is warranted in a future study.

3.5. Coastal Engineering Considerations

The term “seawall” has been utilized throughout this memorandum, although most shoreline stabilization structures are actually referred to as bulkheads. By definition, a seawall is a soil retaining or armoring structure whose purpose is to defend a shoreline against wave attack, whereas a bulkhead is a soil retaining wall structure comprised of vertically spanning sheet piles or other flexural members. The majority of the shoreline in the Village is subject to wave activity, and therefore use of the term “seawall” is generally accurate. The Village shorelines are subject to wave activity from some long fetches.

A source of information on coastal hazards are the flood maps developed by FEMA. The majority of the Village is within an “AE” zone, with base flood elevations ranging from 8 to 10 feet (NGVD) based on 100-year return period event. The FEMA maps for Miami-Dade County are currently being revised, so some of the elevations and/or flood zones may be updated in the near future. By definition, the AE zones have wave heights generally less than 3 feet. Reviewing the FEMA maps, some of the zones change at the shoreline, and there is a high velocity zone (VE) near the northwest area of the Village. Miami-Dade County has performed modeling to identify areas of the Coastal A zone within the county, and most of the shoreline areas within the Village are within this zone (see Figure 3-2).

Figure 3-2: North Bay Village Coastal A Zone (Source: Miami-Dade County and Google Earth)

The Coastal A Zone is within an AE Zone, and delineates an area that may be affected by 1.5-foot or higher breaking waves and may therefore, be at significant risk during a 100-year flood event. While not formally defined in the NFIP regulations or mapped as a flood zone, the area between Zone VE and within the AE zone with limited wave activity is called the Coastal A Zone (see Figure 3-3). This area is subject to flood hazards associated with floating debris and high-velocity flow associated with waves and debris that can erode and scour building foundations and, in extreme cases, cause foundation failure according to FEMA.

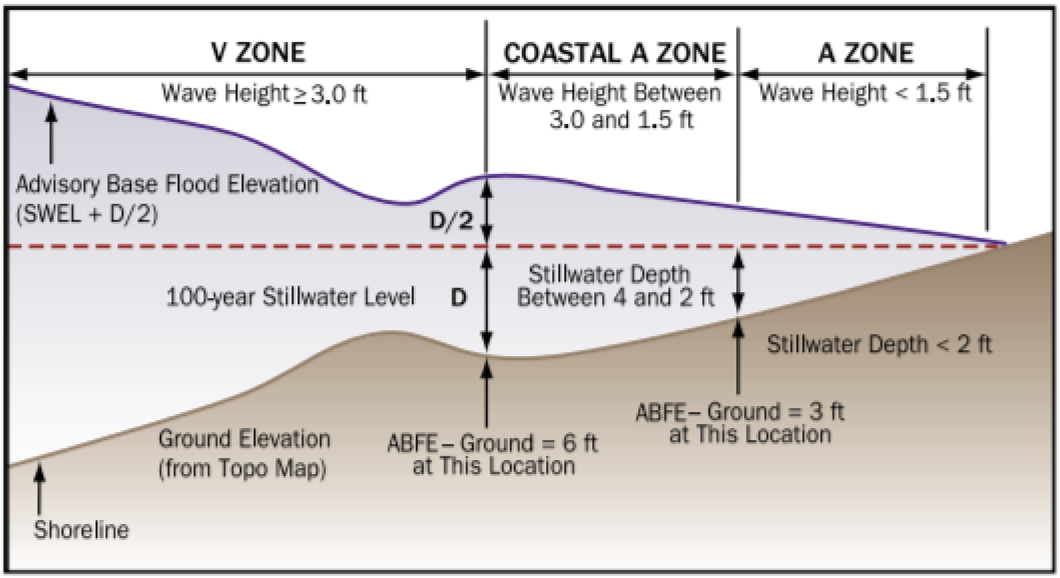

Figure 3-3: Graphic of the Coastal A Zone (Source: FEMA)

Based on this summary information, the shoreline along the Village will be subject to wave attack, not only from the FEMA 100-year storm (1% chance a storm of this magnitude will occur in a year), but also by storms at lower return periods. Depending on the location of the seawall, a site-specific coastal engineering study should be performed as part of the engineering design to evaluate the load cases from waves.

General guidance for the design of seawalls is for areas exposed to 6-foot wave heights should generally be constructed as "gravity" type structures with concave-shaped caps to deflect wave energy and minimize overtopping. Due to the shallow waters of Biscayne Bay, wave heights are not expected to be this high in North Bay Village; therefore, vertical walls can be designed. In addition to geotechnical conditions and live loads for conventional bulkhead design, seawalls need to account for wave pressure/suction, hydrostatic loads due to tidal prism lag, toe scour, overtopping, and seepage. Ideally, new seawalls would be designed for coastal engineering conditions associated with the 100-year return period, however designing for conditions associated with a 50-year event (2% chance of return in any given year) would be more practical and cost effective. Depending on the selected return period, wave pressure and suction forces, immediately prior to overtopping with the storm surge, may result. A common cause of failure in coastal seawalls is the tidal prism lag when during a coastal storm event, extensive wave overtopping with associated rainfall fully saturates the soil behind the wall. As the coastal storm moves through the area, the surface water levels will be lowered at a faster pace than the groundwater behind the wall. This condition with the differential water levels can overstress structural components of a seawall and cause failure. For engineering design purposes, a groundwater equal to that of the mean high water (MHW) is recommended to account for the interconnectivity between the groundwater and the sea level.

3.6. Resiliency Standards

North Bay Village established a Sustainability and Resiliency Task force in January 2019 keen on raising awareness and offering guidance to the Village regarding sustainable design for future development focused on resiliency. The task force has been an integral member in creating a partnership between the Village and the University of Miami's College of Engineering. The College is utilizing and promoting fiber-reinforced polymers for use in seawall construction to inhibit corrosion that often shortens the life of seawalls. These polymers are already being used in bridges built by Florida Department of Transportation (FDOT). Due to the high salinity within Biscayne Bay, the use of new technologies, such as fiber-reinforced polymers instead standard rebar should be considered where feasible, to increase the life of the seawall. Other technologies include the use of seawater in place of freshwater in the concrete mix reducing the impact construction has on the environment.

Resiliency is about developing design criteria that can withstand the test of time, but that is also attainable. By setting the minimum seawall cap elevation at 5.94 ft NAVD (7.50 ft NGVD), North Bay Village will be generally consistent with the surrounding communities of the City of Miami and City of Miami Beach. The community will be more resilient to sea level rise, increased storm surge, and king tides in the future. Based on the latest sea level rise predictions published by the Southeast Florida Regional Climate Change Compact, 40 inches of sea level rise is expected by the year 2070. An increase of 40 inches over the current maximum king tide elevation, brings the predicted tide level in 2070 to 5.60 ft NAVD (7.16 ft NGVD); under the proposed minimum cap elevation. The proposed elevation is also not so high that it would block the views of existing properties on the waterfront. The construction of the seawall at Benihana at elevation 5.94 ft NAVD (7.50 ft NGVD) serves as an example of what building for the future could look like.

3.7. Permitting

In addition to obtaining a building permit from North Bay Village, seawalls will also need to be permitted through the Florida Department of Environmental Protection (FDEP), U.S. Army Corps of Engineers (USACE), and Miami-Dade County Department of Regulatory and Economic Resources (RER) Division of Environmental Resources Management (DERM).

3.7.1. Florida Department of Environmental Protection

The FDEP is the State permitting action agency with jurisdiction over construction activities which take place in, on, over, or upon surface waters of the State and also regulates the use of State-owned submerged lands. The FDEP Environmental Resource Permit (ERP) application review of the proposed Project will primarily focus on impacts to environmental resources, mitigation for those impacts, and the use of State-owned (sovereignty) submerged lands. As the seawalls are proposed on Sovereignty Submerged Lands within the Biscayne Bay Aquatic Preserve, seawalls must be replaced a maximum of 18 inches waterward of the existing seawall wet face. The seawall replacement and riprap required by Miami-Dade County to be placed at the toe of the wall (see more information in subsection 3.7.3) should qualify for a State exemption pursuant to Section 62-330.051(12)(b), Florida Administrative Code (F.A.C.), and Section 373.406(6), Florida Statutes, respectively. These activities should also qualify for a Consent to use sovereignty submerged lands pursuant to Sec. 18-21.005(1)(c), F.A.C. FDEP Form 62-330.050(1) – *"Request for Verification of an Exemption" (June 1, 2018) or a letter that clearly requests an exemption verification and includes pertinent project information should be submitted to the FDEP for review and verification of exemption.*

3.7.2. U.S. Army Corps of Engineers

The USACE is the Federal permitting agency with jurisdiction over all proposed work in or affecting navigable waters and all discharges of dredged or fill material in Waters of the United States. Seawalls and the mitigation riprap that will be required by Miami-Dade County RER DERM to be placed at the toe of the seawalls will require review and approval under Nationwide Permits 3 and 13, respectively. The work must also go through formal consultation with the NOAA National Marine Fisheries Service (NMFS) Office of Protected Resources for issuance of a project-specific Biological Opinion, due to site location within critical habitat for Johnson's seagrass, which is a federally-listed threatened species. If the riprap required by Miami-Dade County DERM to be placed at the toe of the seawall impacts seagrass, the USACE will require mitigation in the form of mitigation bank credit purchase if the mitigation calculations yield a requirement of 0.01 mitigation bank credit or more (a smaller value is exempt). A seagrass survey that is completed during the NMFS-designated growing season of June 1 through September 30 is required for federal agency evaluation relative to unavoidable seagrass impacts and mitigation. A site biological assessment completed by Miami-Dade County RER DERM staff within the federally designated seagrass growing season at the request of the Village or upon submittal of a Class I Permit application and fee will satisfy this requirement.

3.7.3. Miami-Dade RER DERM

The Miami-Dade County RER DERM is the County permitting action agency with jurisdiction over construction activities which take place in, on, over, or upon tidal waters, submerged bay bottom lands, wetlands, or within County canal rights-of-way, reservations or easements in Miami-Dade County. Under Section 24-48 of the Miami-Dade County Code, seawalls will require a Class I Permit from DERM. DERM also requires seawalls within the Biscayne Bay Aquatic Preserve that are of sheet pile construction to be located no more than 18 inches waterward of the existing seawall wet face and requires the top of cap to be a minimum of 6 inches above the adjacent upland grade elevation for stormwater runoff management. DERM will require 1 cubic yard (CY) of riprap per linear foot of new/replacement bulkhead as mitigation for water quality impacts associated with the vertical structure. An 8-foot-wide swath of riprap is required to be placed at the toe of new/replacement seawalls at a 2:1 slope, even if this creates seagrass impacts for which the USACE will require mitigation. If the total required volume of riprap does not fit onsite, which it often does not due to shallow water depths, the balance of the mitigation is typically addressed in the form of a monetary contribution to the Biscayne Bay Environmental Enhancement Trust Fund. The requirements of the Miami-Dade County Erosion Control Line (ECL) do not apply in the area of North Bay Village. The ECL only applies to oceanfront seawalls only.

It will be the permittee's responsibility to obtain any and all necessary permits that may be required by any other government agencies.

4. Seawall Alternatives

Outlined in this section are the four main alternatives for seawall construction applicable and acceptable to North Bay Village. Each alternative is proposed to be installed no greater than 18 inches from the wet face of the existing seawall, have a grade tie-in six inches below the top of cap, and have one cubic yard per foot of seawall of riprap included, as required by permitting agencies. Property owners can evaluate the permitting feasibility of extending the seawall further waterward; however, permitting in Miami-Dade County can be challenging and the Village is within the Biscayne Bay Aquatic Preserve. Individual property owners will be responsible for ensuring connectivity with adjacent seawalls, keeping in mind return walls may be required for some locations. The cap is set at a minimum elevation of 5.94 ft NAVD (7.50 ft NGVD) for all alternatives. It will be the responsibility of the design engineer to determine applicability at specific site locations and perform a full engineering design of the seawall. The typical sections shown herein are for guidance only.

These alternatives were developed based on local experience, typical geotechnical conditions, availability of materials, and local contractor capabilities. While the various walls will vary in appearance from the waterside, they will look uniform from the landside, except for the living shoreline, as all options proposed utilize a concrete cap.

4.1. Concrete Seawall

Concrete pile and panel seawalls, whether in conjunction with batter piles (Figure 4-1) or tie-backs (Figure 4-2), are widely used and accepted throughout South Florida. Based on the geotechnical conditions outlined in this report, construction of concrete seawalls is expected to range in cost from $2,200 to $3,100 per linear foot. This cost does not include permitting but does include the cost of the riprap.

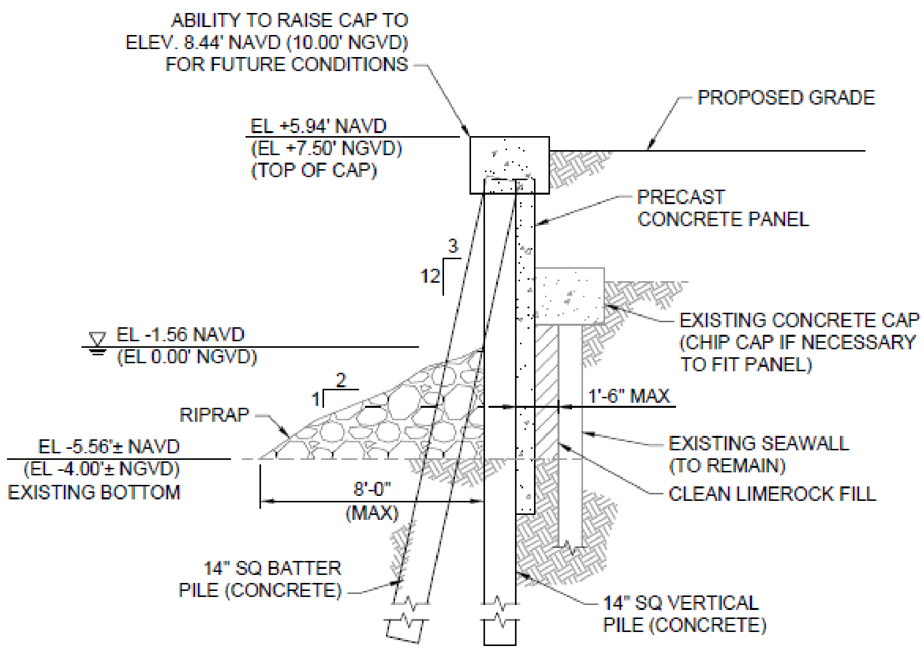

Figure 4-1: Concrete Seawall with Batter Pile Typical

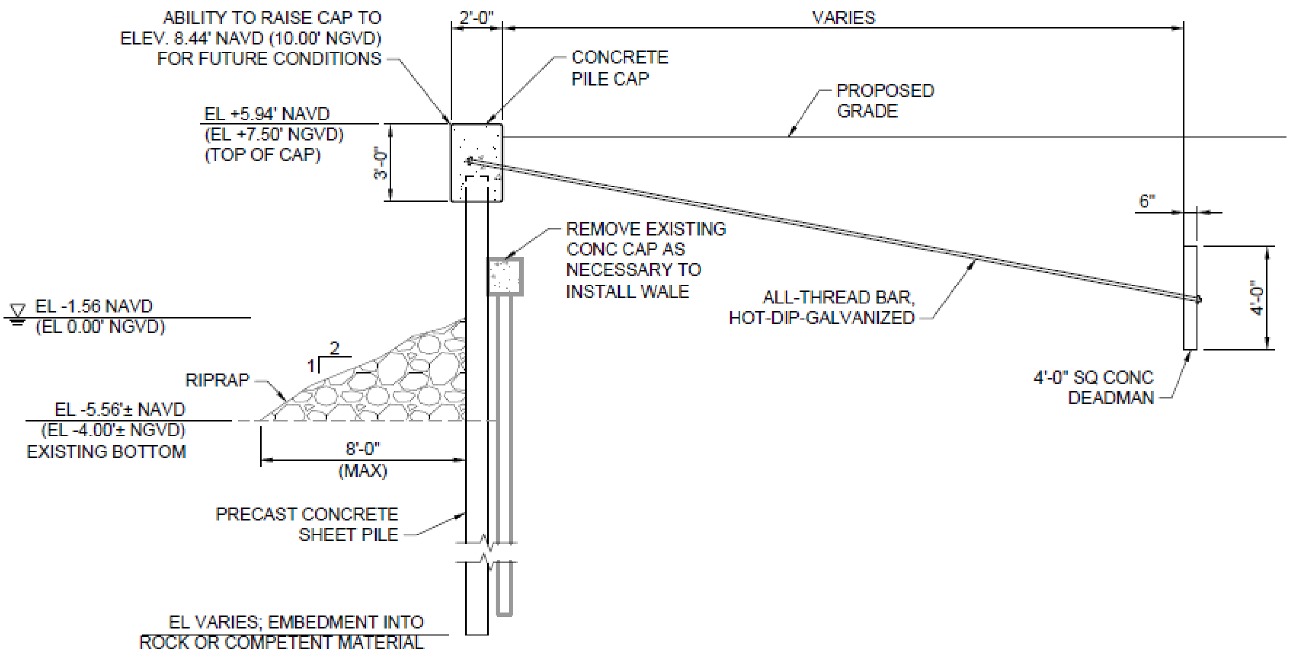

Figure 4-2: Concrete Seawall with Tie-back and Deadman Typical

4.2. University of Miami GFPR Concrete Seawall

The Glass Fiber Polymer Reinforced (GFPR) Seawall, pioneered by University of Miami's College of Engineering, is similar in design to the standard concrete seawalls shown in Section 4.1, with the exception of the use of GFPR instead of the standard steel rebar and tie rods. It also has the ability to use concrete made with seawater versus freshwater, thus improving the sustainability of this design. Seawall components made with seawater will need adequate upland area to perform concrete mixing and be cast-in-place. Precast elements are not available in seawater concrete, at this time. Figure 4-3 shows the GFPR concrete batter pile wall and Figure 4-4 shows the GFPR concrete wall with tie-backs. The use of GFPR instead of steel reinforcement increases the cost per linear foot by about 10 percent. Seawalls constructed with GFPR are expected to have a 50-year design life. The use of GFPR reduces the rate of corrosion. GFPR is already being used by FDOT in the design on concrete structures in highly aggressive environments. Appendix D contains additional information from University of Miami on the testing and design of the GFPR, as well as other progressive materials used in seawall design.

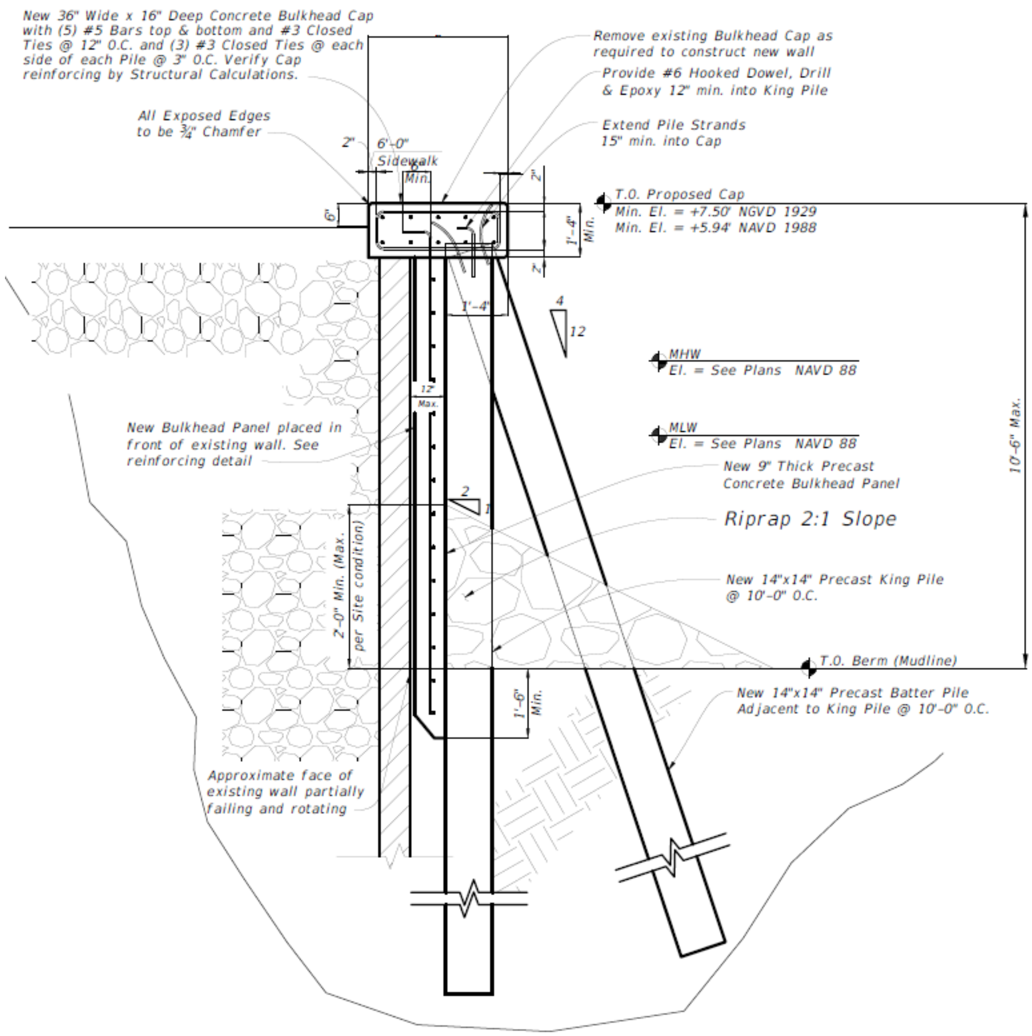

Figure 4-3: University of Miami's GFPR Concrete Seawall with Batter Pile (Courtesy of University of Miami's College of Engineering)

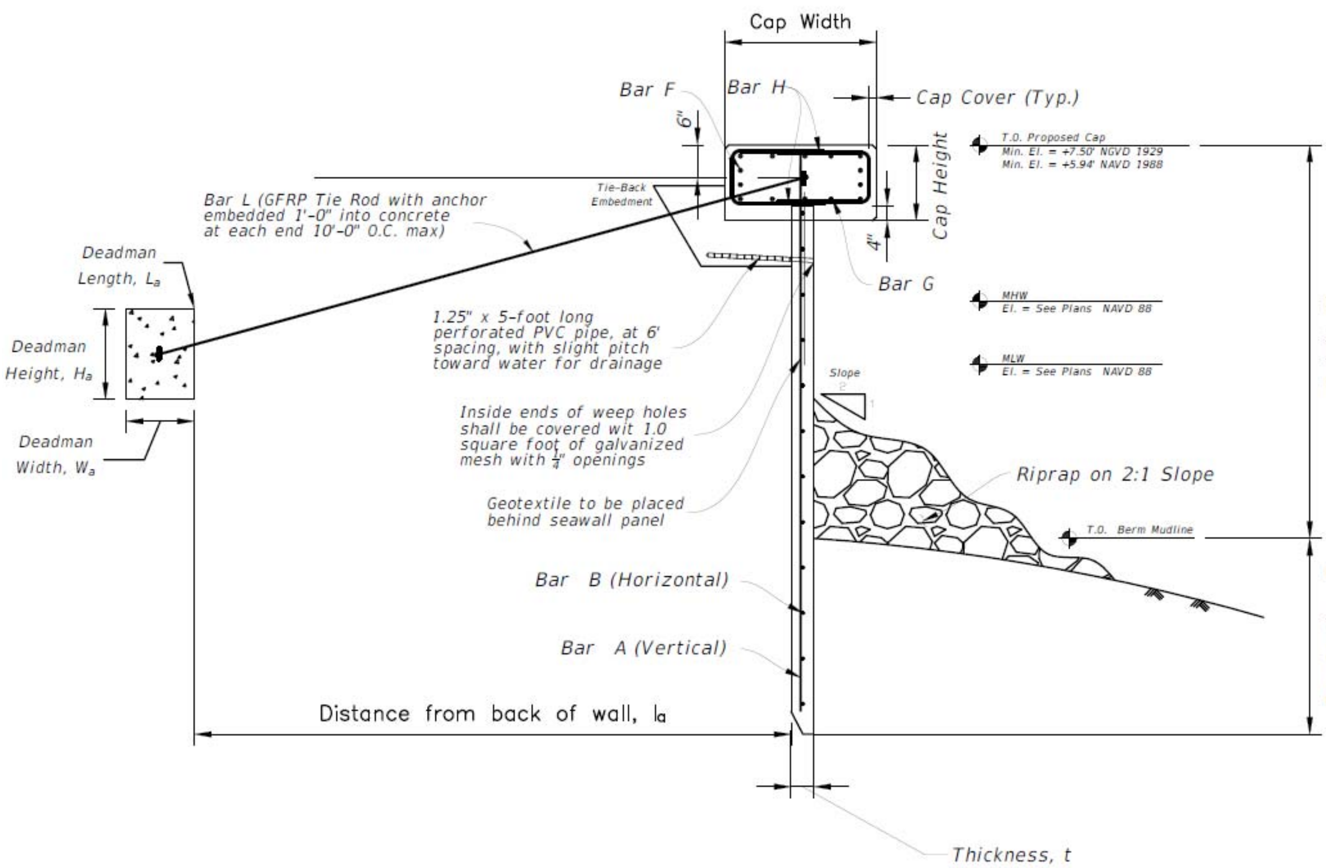

Figure 4-4: University of Miami's GFPR Concrete Seawall with GFPR Tie-back (Courtesy of University of Miami's College of Engineering)

4.3. Sheet Pile Seawall

Sheet pile seawalls can be steel, composite, or vinyl. Steel sheet pile walls, with and without tie-backs, are often used in South Florida for their ability to penetrate the hard limestone layer without the need for predrilling (see Figure 4-5). Composite sheet piles (see Figure 4-6) are more widely used in other parts of the state; based on the geotechnical information obtained in Section 3.2 and the anticipated exposed height, they are a viable option for North Bay Village. Vinyl sheet piling is not applicable due to the exposed wall height. Construction of steel sheet pile walls is expected to range in cost from $2,900 to $3,800 per linear foot, while composite sheet pile is expected to range from $1,800 to $2,800 per linear foot installed. This cost does not include permitting but does include the cost of the riprap.

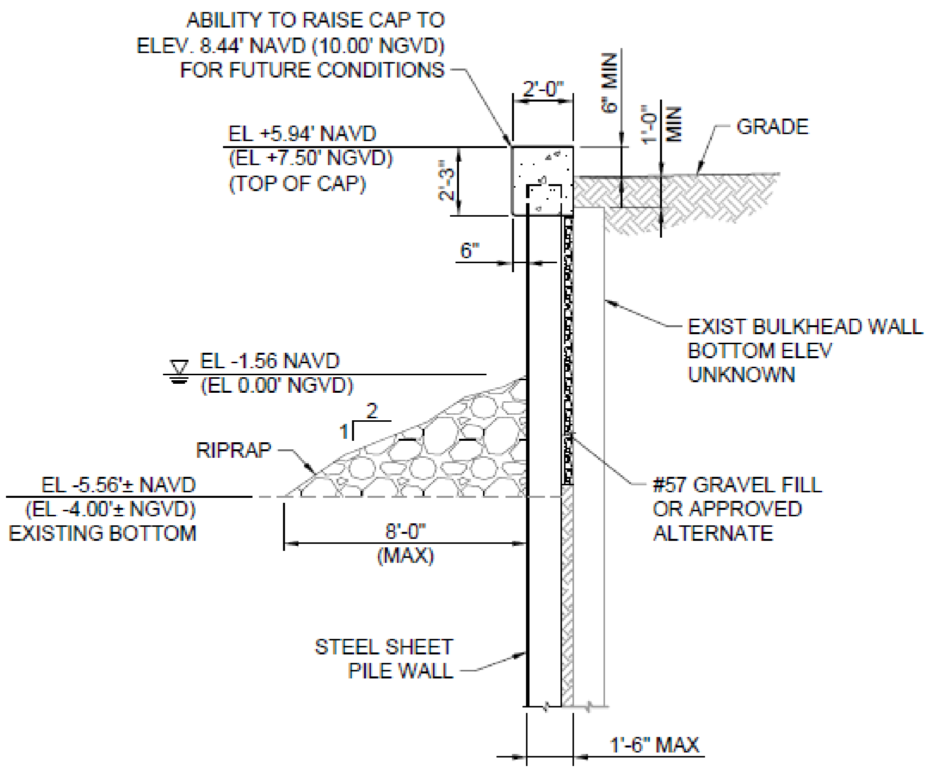

Figure 4-5: Cantilever Steel Sheet Pile Wall Typical

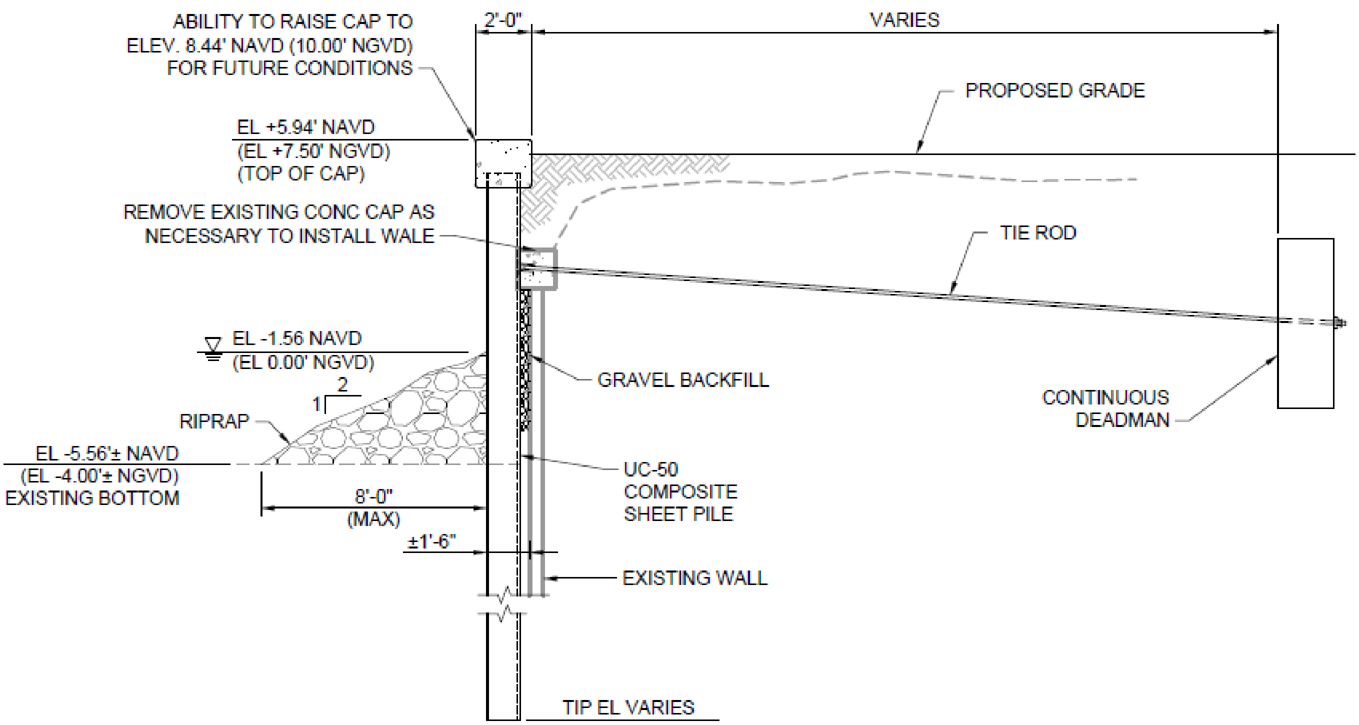

Figure 4-6: Composite Sheet Pile Wall with Tie-back and Deadman Typical

4.4. Living Shoreline

Living shorelines can be a natural solution to protecting uplands from the impacts of sea level rise and storm surge. They also create/restore beneficial habitats for the tidal ecosystem. A living shoreline typically involves the placement of material beyond the property line on submerged land in the riparian right-of-way. Unfortunately, due to permitting, living shorelines with Miami-Dade County are not allowed to encroach upon this submerged land. However, property owners may elect to construct a living shoreline on their own property if the toe of the wall is not advanced seaward from its current location and adequate height and protection to abutting properties are provided.

Figure 4-7 shows a possible living shoreline option that would limit the upland excavation needed. This section shows the riprap extending 8 ft beyond the existing wall to match the toe of adjacent riprap to reduce the upland impacts; however, the portion of the riprap beyond the wall may be difficult to permit. Mangroves would be planted to provide stabilization and additional ecological benefit. Special permits may be obtained for trimming planted vegetation to limit the view impacts to the upland owner. Construction cost of the riprap ranges from approximately $220 to $290 per cubic yard. The volume of riprap needed can vary greatly from site to site. This cost does not include the permitting or the planting of vegetation.

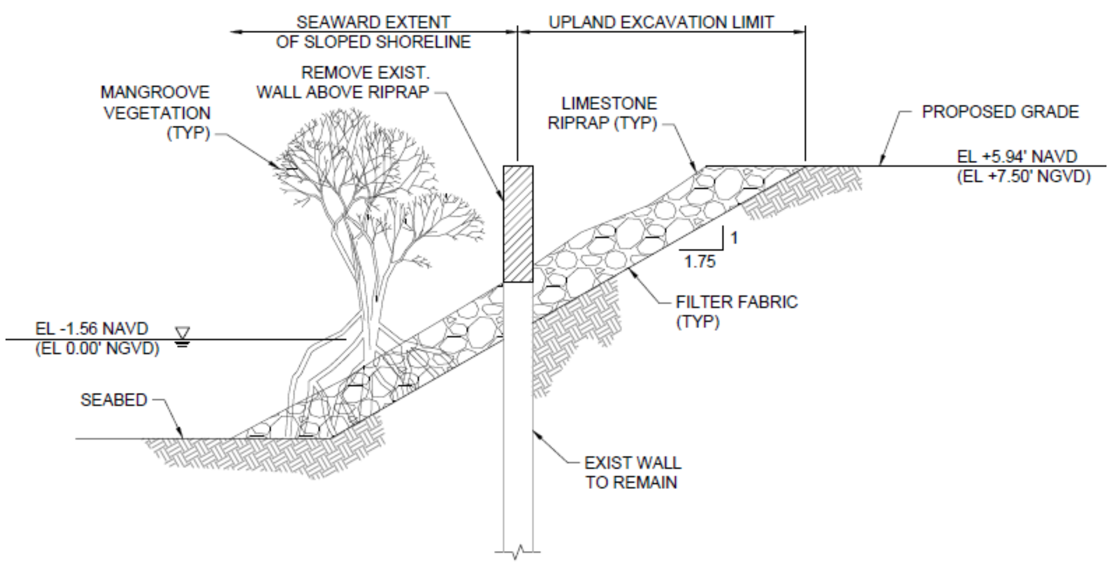

Figure 4-7: Living Shoreline Typical with Mangrove Plantings

Various additional living shoreline options are also available on the market today that utilize a smaller footprint than was historically feasible. For example, several geosynthetic vendors manufacture grid systems for marine applications (see Figure 4-8). Vegetated Reinforced Soil Slopes can also be installed (see Figure 4-9).

Figure 4-8: Tensar Triton Marine Cells (courtesy of www.tensarcorp.com)

Figure 4-9: Tensar Vegetate Reinforced Soil Slopes (courtesy of www.tensarcorp.com)

4.5. Raising an Existing Wall

In addition to seawall replacement, it may be possible to raise the existing seawall in select locations throughout the Village to meet the new required height of 5.94 ft NAVD (7.50 ft NGVD). Individual owners will need to obtain a full structural evaluation from an engineer prior to design and approval of raising any existing seawall. Figure 4-10 is a cross section of a typical section with a raised cap.

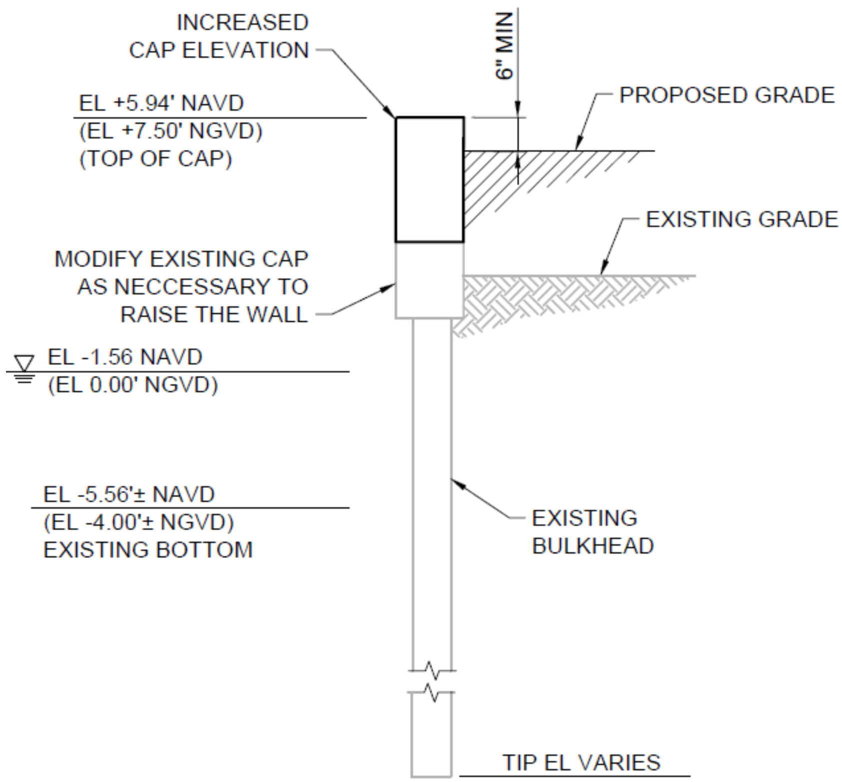

Figure 4-10: Existing Wall with Raised Cap Typical

4.6. Return Walls

Individual property owners will be required to tie-in to the surrounding seawalls as they raise or replace the existing walls. Return walls may be required in some locations. Return walls at the 90-degree section of the seawalls that turn into one's property when an adjacent wall cannot be tied into. This will need to be analyzed on a case-by-case basis. Figure 4-11 shows a typical return wall with a concrete cap.

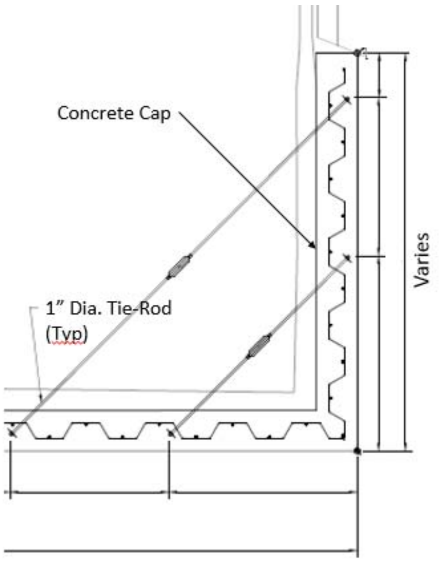

Figure 4-11: Typical Return Wall with Tie-rods and Concrete Cap Plan View and Photo

5. Conclusions and Recommendations

North Bay Village is actively facilitating overall resiliency by adapting to rising sea levels and providing increased coastal defense through the construction of higher seawalls. These criteria will be adopted into a municipal ordinance for adoption into the Village code. The background information outlined in this memorandum emphasizes the necessity of this code development. With as little as 3 feet of sea level rise, which is predicted to happen in less than 50 years, a large portion the Village will experience flooding. With the establishment of code outlining the minimum seawall criteria, North Bay Village will be able to slowly increase its resiliency.

Cap elevations for recent seawall projects within the Village range from 2.64 ft NAVD (4.20 ft NGVD) to 5.94 ft NAVD (7.5 ft NGVD). Surrounding communities within Broward and Miami-Dade Counties have varying seawall elevation criteria. The two most stringent ordinances are for the City of Miami Beach and the City of Miami. The City of Miami Beach has a minimum cap elevation for public properties set at 5.70 ft NAVD (7.26 ft NGVD), while the proposed ordinance for the City of Miami will set the minimum cap elevation at 6.0 ft NAVD (7.55 ft NGVD). After reviewing the available data and past projects and evaluating the standards set in adjacent communities, the minimum seawall elevation is recommended to be established at 5.94 ft NAVD (7.5 ft NGVD). The seawall designs will also need to account for an additional 2.5 ft of cap elevation to be raised in the future without the need for redesign.

Various seawall design options are applicable to the shoreline of North Bay Village, including concrete pile and panel walls, University of Miami's Glass Fiber Polymer Reinforced Concrete Wall, and steel or composite sheet pile walls, as well as living shorelines. Seawalls proposed for the Village should be designed and constructed in accordance with one of these configurations. The walls are expected to range in cost from $1,800 to $3,800 per linear foot; however, each seawall will be site-specific and will need to account for localized conditions including adjacent walls. The seawalls should also be designed for the additional cap elevation, as well as for coastal engineering conditions referenced in this memo. The design with supporting calculations should demonstrate the proposed seawall meets the requirements of the Village.

Sea level does not happen independent of groundwater rise. The latest study shows a one-to-one relationship between sea level rise and groundwater rise in coastal Miami-Dade County. Seawalls alone, will not protect the island community from rising seas, but it will make the community more resilient against some King Tides as well as against wave action and storm surge.

Seawall Code Recommendations:

- Minimum Cap Elevation
    - 5.94 ft NAVD (7.50 ft NGVD) with ability to raise cap additional 2.5 ft in future
- Coastal Engineering Considerations
    - Wave impact
    - Storm surge
    - Hydrostatic loads due to tidal prism lag
    - Toe scour
    - Overtopping
    - Seepage
- Minimum Design Service Life
    - 25 Years
- Allowable Seawall Configurations
    - Concrete Pile and Panel Walls
    - University of Miami's Glass Fiber Polymer Reinforced Concrete Wall
    - Sheet Pile Wall
        - Steel
        - Composite
    - Living Shorelines.

The recommendations in this technical memo will need to be incorporated into an ordinance developed by the Village for implementation into the Village Code. The code should address proposed new public and private seawalls. The planning for the Island Walk project will need to account for this updated seawall criteria and will set a standard for the Village. The Village will need to consider the requirement for existing private property owners to upgrade their existing seawalls to these new requirements within an established period of time. For example, the Broward County ordinance requires all seawalls that are new or have substantial improvements to immediately comply with the revised code, while damaged or structurally deficient seawalls, will have a period of 365 days from notification from the County to repair or replace the seawall and bring it into compliance with the new code. The current threshold for required replacement is defined as having a repair cost greater than 50% of the current replacement cost. Some municipalities are considering special taxing districts, bonds, or other sources of funding to replace seawalls in a holistic manner. Advantages to this approach include an enhanced and consistent seawall system that is planned for the future and can provide more immediate coastal storm and king tide protection. The Village would receive more competitive and cost-effective approaches to construction for a larger scale project.

The U.S. Army Corps of Engineers is progressing with the Miami-Dade Back Bay Coastal Storm Risk Management (CSRM) Study. This study is a comprehensive feasibility study to promote resiliency and reduce the risk of coastal storm damage within the County. The study will develop and evaluate implementable CSRM structural, nonstructural, and natural and nature-based feature measures for the County which will be formulated to reduce risk to residents, industries, businesses, and infrastructures which are critical to the nation's economy. The County has high levels of risk and vulnerability to coastal storms which will be exacerbated by combinations of sea level rise, saltwater intrusion, and climate change. The CSRM has resulted in design concepts for several areas of the County, and the area along Edgewater in the City of Miami consists of a large-scale floodwall. If any of these projects move forward, some of this design and construction would require eminent domain to obtain properties. The Corps would fund these projects 65% with local governments, including the County, funding the remaining local share. Evaluating a large-scale flood wall, similar to the system proposed for the Edgewater area, is beyond the scope of this technical memo. Updated concepts are due in a few months from this $3M CSRM study funded by the Corps, with final concepts due mid-year 2021. The Village should continue to monitor the outcome of this study; however, at this time, North Bay Village is not included in the area to be protected by the floodwall.

6. References

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Appendix A

Benihana Seawall and Treasures on the Bay Seawall Design Plans

RECEIVED
NOV 02 2017
DERM Coastal Resources Section
Natural Resources Regulation & Restoration
Division (NRRRD)

NORTH BAY VILLAGE

ZONING DEPARTMENT
BUILDING PERMIT REVIEW

ADDRESS: 1665 Kennedy Causeway

OWNER: COLE BN NORTH BAY VILLAGE FL LLC

PROJECT TYPE: Seawall Repair/Replacement

DATE: October 25, 2017

REVIEW HOURS: 1.0

APPROVAL/DENIAL: Approved

COMMENTS:

These plans for the replacement of a seawall with new sheet piles, new batter piles, new cap, and limestone riprap are approved, as recommended in the attached seawall report, as submitted by the applicant and as attached to this document.

ZONING OFFICIAL: James G. LaRue, AICP

SIGNATURE: James G. LaRue

North Bay Village
Building Department
1666 Kennedy Causeway #101, North Bay Village, FL 33141
Tel: (305) 754-6740 Fax: (305) 756-7722

November 4, 2016

Cole BN North bay Village FL LLC
Benihana Inc.
21500 Biscayne Blvd.
Suite # 900
Aventura, FL 33180

Re: Seawall and Bulkhead Maintenance and Repair
1665 Kennedy Causeway
North Bay Village, FL 33141

Dear Property Owner:

This letter is to confirm that the Building Official has reviewed the documentation submitted by Glen Larson (E410) of Dock and Marine Construction Corp with regards to the above referenced seawall and bulkhead.

The letter submitted indicates that the seawall and/or bulkhead is not in satisfactory structural condition and repairs will be required.

A proper building permit must be obtained within 90 days of the date of this letter, and repairs completed within 150 days of permit issuance.

If you have any questions, please contact the Building Department at (305) 754-6740 Monday through Friday between the hours of 8:00 A.M. and 4:00 P.M. Thank you for your cooperation in this matter.

Sincerely,

M Arronte

Miguel Arronte, Chief Building Inspector
c/o Raul Rodriguez, Chief Building Official

RECEIVED
NOV 02 2017
DERM Coastal Resources Section
Natural Resources Regulation & Restoration
Division (NRRRD)

90 Days
150 Days } MM
11/4/16

RECEIVED
NOV 03 2016
By______

Dock and Marine Construction Corp.

752 N.E. 79th Street
Miami FL 33138
O:(305)751-9911 F:(305)-751-4825 john@dockandmarine.net

October 31, 2016

The seawall at 1665 79th Street Causeway was inspected at low tide on 10-22-16. There is aproximatley 260 Linear Feet of water frontage. There are various locations throughout the entire length of seawall which display both vertical and horizontal cracking. Existing King piling are also displaying cracks throughout the entire length of the structure. Due to the age of the existing seawall combined with the evidence found during the inspection we highly recommend the instillation of a new seawall. The new seawall should be engineered as to support the garden and parking lots (Aproximatley 7 to 7.5 NGVD).

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DERM Coastal Resources Section
Natural Resources Regulation & Restoration
Division (NRRRD)

STATE CRC032121

BROW 87-963
P.B. U16718

Ray Qualmann
MARINE CONSTRUCTION INC

ESTABLISHED IN 1948

P.O. BOX 1487 • POMPANO BEACH, FLORIDA 33061
(954) 941-0132 • FAX (954) 943-6430

PROPOSAL

NAME: Benihana, Inc., Attn: Jennifer Cristal
ADDRESS: 21500 Biscayne Blvd., Suite 900
Aventura, FL 33180

DATE: January 13, 2017
CELL:
PHONE: 305-702-2802
FAX: 305-675-3849
E-MAIL: jcristal@benihana.com

LOCATION OF PROJECT:
Same as above

Ray Qualmann Marine Construction, Inc. ("Contractor") proposes to furnish labor, equipment and material in accordance with plans and or specifications provided by customer for the above job, as follows:

Construct (262) two hundred and sixty two lineal ft. of new steel sheet pile seawall no more than 12" water ward of the existing wall as follows:

Furnish and install (262) two hundred and sixty two lineal ft. of 20' long epoxy coated **SCZ – 14** steel sheet pile.
Furnish and install (27) twenty seven 12"x12" prestressed concrete batter pile placed on a 20 degree angle 10' O.C.
Construct (262) two hundred and sixty two lineal ft. of 14" thick 36" wide 5,000 PSI reinforced concrete cap.
Backfill between the new and existing wall with clean fill.
NOTE: New wall to be constructed at the same elevation as the top of the existing retaining wall.
COST: $256,760

Furnish and install approx. (262) two hundred and sixty two cubic yards of limestone rip rap rock placed on a 2:1 slope in front of the new wall as required by DERM for all new seawalls.
COST: $23,580
SUB TOTAL: $280,340

PLEASE NOTE THE FOLLOWING:

- All plans and permit fees applied for by Qualmann Marine will be billed and paid for by customer as an additional expense. **(Estimated cost $20,000).**
- No plumbing, electrical, sod, or landscaping work is included.
- Owner shall be responsible for furnishing rock Mitigation Bond (estimated cost $18,000) to Dade County Derm and returned upon completion of work.
- An existing PVC sewer pipe through the seawall may have to e addressed if requested by DERM.
- When returning your signed contract, please include a copy of survey for permitting.

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Division (NRRRD)

TERMS:

- $20,000 upon acceptance of contract for permitting.
- $200,000 upon placement of steel sheet pile excluding cap.
- Balance upon completion of work.

Initials AH

Ray Qualmann

All work will be guaranteed for one (1) year from the date of completion against defective materials or workmanship. Any changes from the above specifications involving extra costs, after acceptance of the Proposal, will be implemented only upon receipt of a written order and will become an extra charge in addition to the amount of this Proposal. The Customer shall be responsible for furnishing offset stakes set by a licensed surveyor. Contractor is held harmless for claims based upon noise and/or vibration damage. Contractor does not assume any responsibility for permits, plans or engineering, unless otherwise specified herein. Contractor reserves the right to withdraw this Proposal if not accepted in writing within thirty (30) days unless otherwise specified herein.

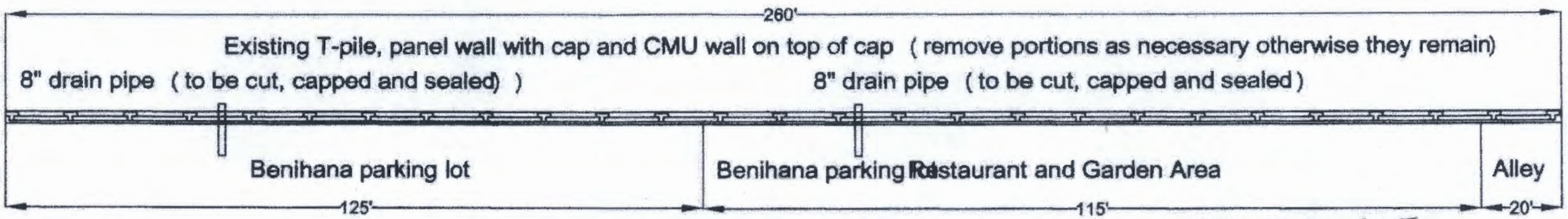
TERMS: Payment in full is due upon completion if not otherwise specified herein. All sums not paid when due shall bear interest at the highest rate allowed by law. If Contractor is required to collect any amounts due by litigation or after demand by an attorney, the Customer shall be obligated to pay all costs plus attorney's fees incurred by Contractor.

ACCEPTED:
By: Adam Wlavaty
Date: 1/18/17

Ray Qualmann Marine Construction, Inc
By: Mitchell Scavone
Date: January 13, 2017

If proposal is accepted please sign and return (1) copy which will be our order to proceed with work and will constitute the entire contract between Customer and Contractor. No verbal agreements shall be part of this contract or binding unless reduced to writing and signed by all parties.

Initials____

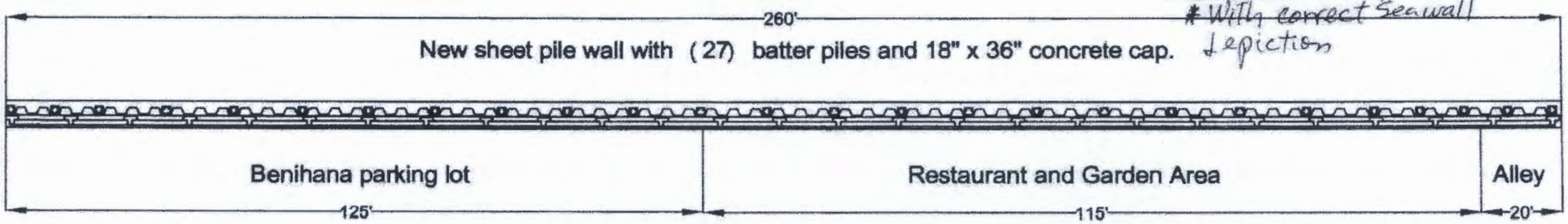


CLI-2017-0115
RER - NATURAL RESOURCES DIVISION
PRELIMINARY APPROVAL
NAME
DATE 7/6/17

Existing Site Plan n.t.s.

New sheet pile wall shall be located within 18" of existing seawall panel

\# With correct Seawall depiction



Proposed Site Plan n.t.s.

General Notes:
1. All concrete to be 5000 psi, 28 day comp. strength.
2. All steel rebar to be Grade 60.
3. Concrete shall be properly cured.
4. Contractor to obtain utility clearance prior to pile driving. Call 1-800-432-4770

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Division (NRRRD)

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JUN 30 2017
DERM Coastal Resources Section
Natural Resources Regulation & Restoration
Division (NRRRD)

ALAN W. GARCIA
LICENSE
No. 42564
STATE OF FLORIDA
PROFESSIONAL ENGINEER

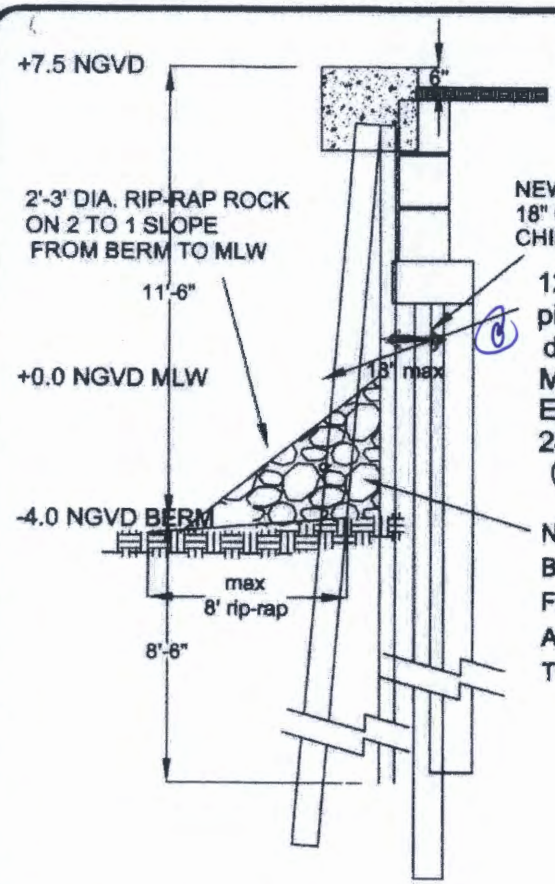
6/28/17
Alan W. Garcia, PE
Fla. Reg. #42564

Owner: BENIHANA
1665 NE 79th ST CSWY

Contractor: Qualmann Marine

Engineer: Alan W. Garcia, P.E.
4202 NW 54th Street
Coconut Creek, FL 33073

New Seawall Site Plan
Sheet 2 of 4
Design By: AWG Drawn By: AWG Checked By: AWG



EXISITING WALL MADE OUT OF CMU
BLOCK ON TOP OF T-PILE PANEL WALL
WALL TO REMOVED AS NEEDED

NEW WALL TO BE PLACED WITHIN
18" OF EXISTING SEAWALL PANEL
CHIP EXISTING CAP IF NECESSARY TO FIT PANEL

12" x 12" Concrete, 10' o.c.
pile (Cap. 34 Kips) on 18 deg batter
driven min. 15' embeddment.
Min. 20 ton bearing capacity
Extend pile prestressed strands
24" into slab or epoxy dowel
(4) #5 into piles.

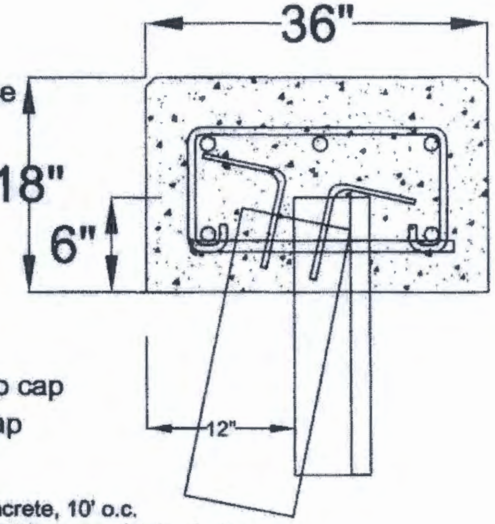
NEW PZ-22 20' STEEL SHEETS
BACKED WITH
FILTER FABRIC
AND EPOXY COATED 16 MIL
TOP 14' BOTH SIDES

CROSS-SECTION OF SEAWALL PANELS n.t.s.

(5) #5 BARS CONT.
#4 STIRRUPS, 12" O.C.
Burn hole thru panel and place
#8 at each stirrup location

CL1-2017-0115

RER - NATURAL RESOURCES DIVISION
PRELIMINARY APPROVAL
NAME
DATE 7/6/17



Panel embedded 12" into cap
Pile embedded 6" into cap

RECEIVED
JUN 30 2017
DERM Coastal Resources Section
Natural Resources Regulation & Restoration
Division (NRRRD)

12" x 12" Concrete, 10' o.c.
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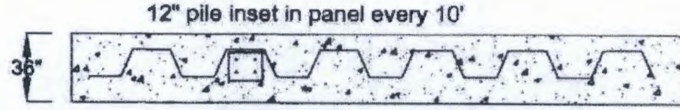
Extend pile prestressed strands
24" into slab or epoxy dowel
(4) #5 into piles.

CAP DETAILS n.t.s.

General Notes:
1. All concrete to be 5000 psi, 28 day comp. strength.
2. All steel rebar to be Grade 60.
3. Concrete shall be properly cured.
4. Contractor to obtain utility clearance prior to pile driving. Call 1-800-432-4770.

12" pile inset in panel every 10'



PILE/PANEL LAYOUT n.t.s.

ALAN W. GARCIA LICENSE No. 42564 STATE OF FLORIDA PROFESSIONAL ENGINEER

6/28/17
Alan W. Garcia, PE
Fla. Reg. #42564

Owner: BENIHANA
1665 NE 79th ST CSWY

Contractor: Qualmann Marine

Engineer: Alan W. Garcia, P.E.
4202 NW 54th Street
Coconut Creek, FL 33073

New Seawall
Sheet 3 of 4
Design By: AWG Drawn By: AWG Checked By: AWG

RECEIVED
NOV 02 2017
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Natural Resources Regulation & Restoration
Division (NRRRD)

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RER - NATURAL RESOURCES DIVISION
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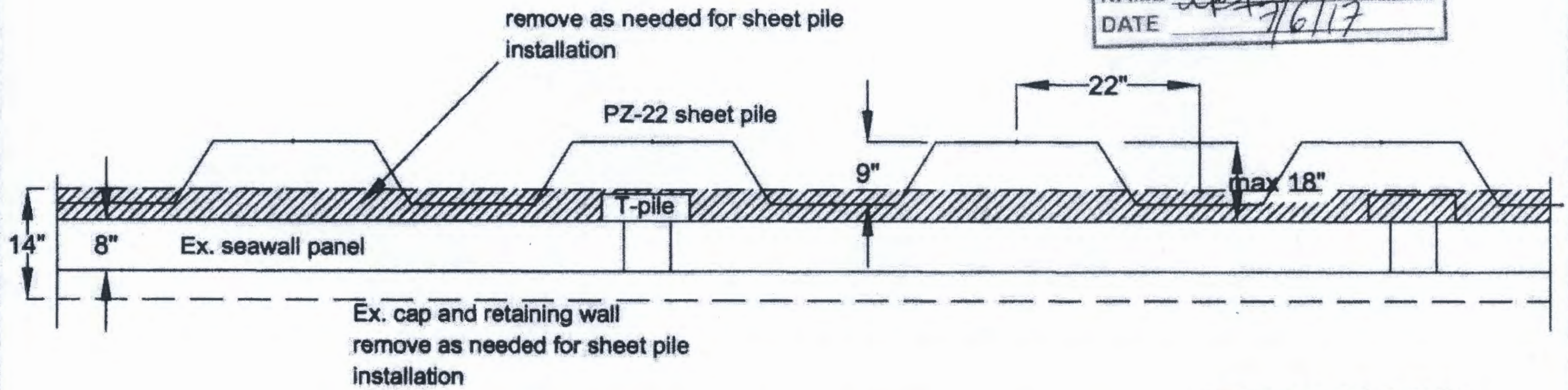

Placement of sheet pile panels within 18" of ex. seawall panel

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Natural Resources Regulation & Restoration
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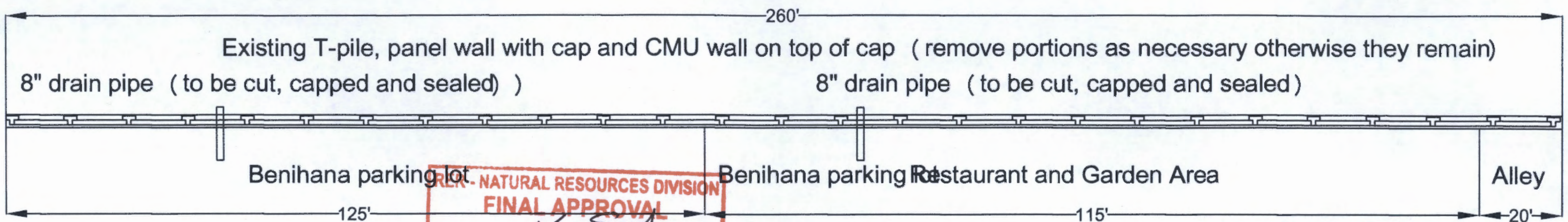
6/28/17
Alan W. Garcia, P.E.
Fla. Reg. #42564

Owner: BENIHANA
1665 NE 79th ST CSWY

Contractor: Qualmann Marine

Engineer: Alan W. Garcia, P.E.
4202 NW 54th Street
Coconut Creek, FL 33073
(954) 304-0098

Sheet Pile Placement
Sheet 1 of 1
Design By: AWG Drawn By: AWG Checked By: AWG



RER - NATURAL RESOURCES DIVISION
FINAL APPROVAL
NAME N.S.
DATE 11/15/17

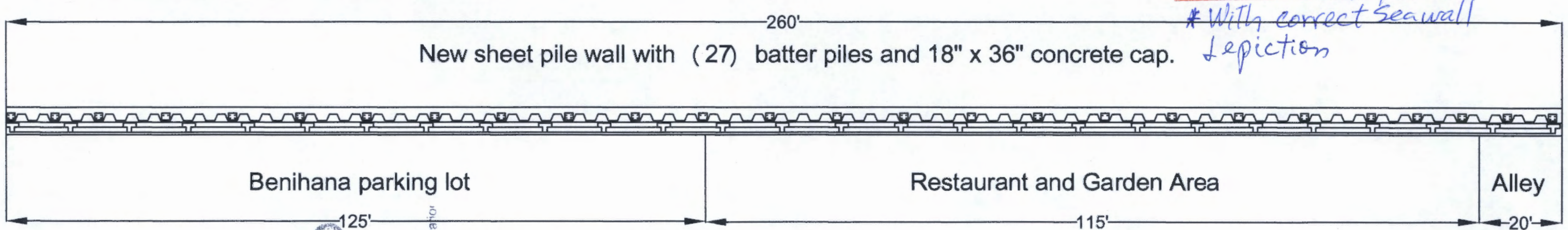
Existing Site Plan n.t.s.

CLI-2017-0115

RER - NATURAL RESOURCES DIVISION
PRELIMINARY APPROVAL
NAME
DATE 7/6/17

* With correct seawall depiction

New sheet pile wall shall be located within 18" of existing seawall panel



Proposed Site Plan n.t.s.

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ALAN W. GARCIA LICENSE No. 42564 STATE OF FLORIDA PROFESSIONAL ENGINEER

6/28/17
Alan W. Garcia, PE
Fla. Reg. #42564

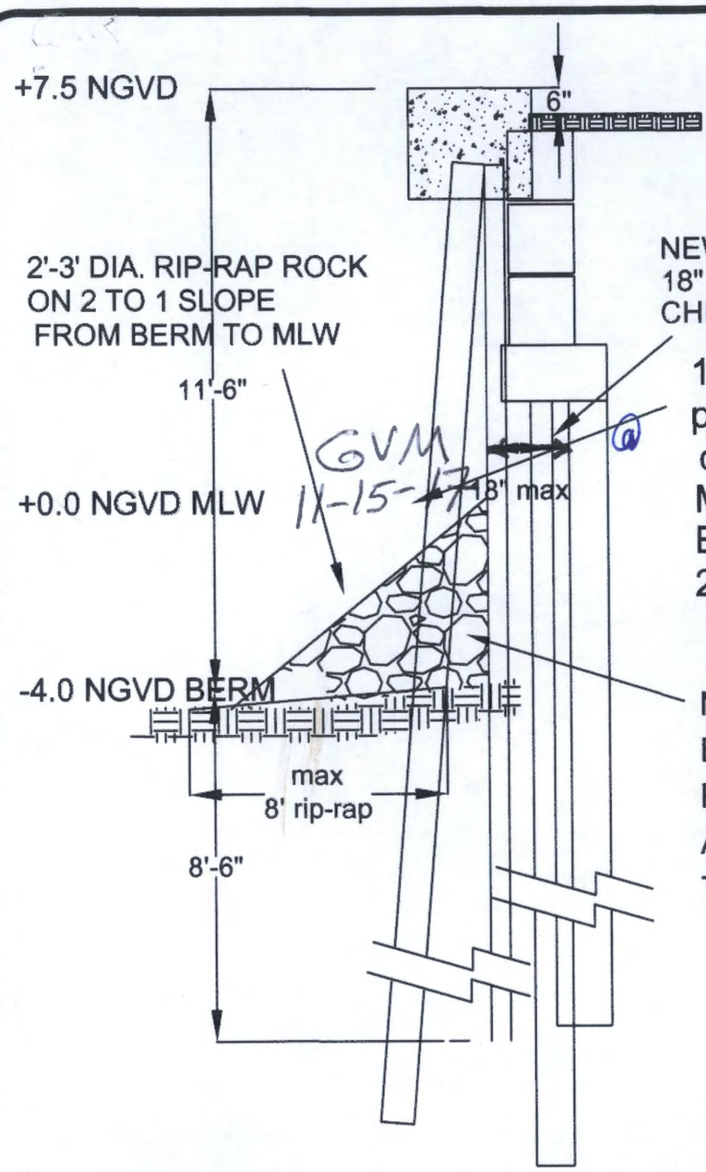
Owner: BENIHANA
1665 NE 79th ST CSWY

Contractor: Qualmann Marine

Engineer: Alan W. Garcia, P.E.
4202 NW 54th Street
Coconut Creek, FL 33073

New Seawall Site Plan
Sheet 2 of 4

Design By: AWG Drawn By: AWG Checked By: AWG



EXISITING WALL MADE OUT OF CMU BLOCK ON TOP OF T-PILE PANEL WALL WALL TO REMOVED AS NEEDED

NEW WALL TO BE PLACED WITHIN 18" OF EXISTING SEAWALL PANEL CHIP EXISTING CAP IF NECESSARY TO FIT PANEL

12" x 12" Concrete, 10' o.c. pile (Cap. 34 Kips) on 18 deg batter driven min. 15' embeddment.
Min. 20 ton bearing capacity
Extend pile prestressed strands 24" into slab or epoxy dowel (4) #5 into piles.

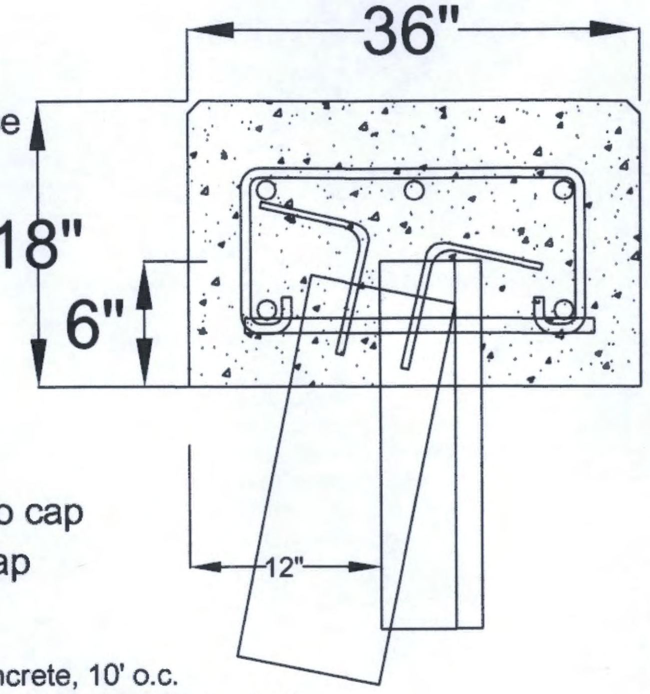
NEW PZ-22 20' STEEL SHEETS BACKED WITH FILTER FABRIC AND EPOXY COATED 16 MIL TOP 14' BOTH SIDES

CROSS-SECTION OF SEAWALL PANELS n.t.s.

(5) #5 BARS CONT.
#4 STIRRUPS, 12" O.C.
Burn hole thru panel and place #8 at each stirrup location

CL1-2017-0115

RER - NATURAL RESOURCES DIVISION
PRELIMINARY APPROVAL
NAME [signature]
DATE 7/6/17



Panel embedded 12" into cap
Pile embedded 6" into cap

RECEIVED
JUN 30 2017
DERM Coastal Resources Section
Natural Resources Regulation & Restoration Division (NRRRD)

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RER - NATURAL RESOURCES DIVISION
FINAL APPROVAL
NAME [signature]
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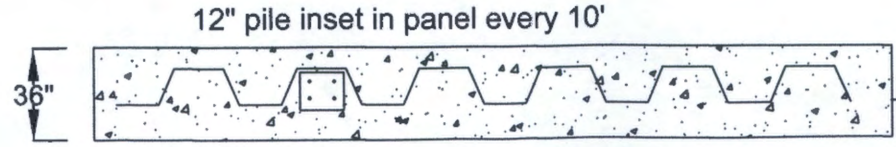

PILE/PANEL LAYOUT n.t.s.

ALAN W. GARCIA LICENSE No. 42564 STATE OF FLORIDA PROFESSIONAL ENGINEER
[signature] 6/28/17
Alan W. Garcia, PE
Fla. Reg. #42564

Owner: BENIHANA
1665 NE 79th ST CSWY

Contractor: Qualmann Marine

Engineer: Alan W. Garcia, P.E.
4202 NW 54th Street
Coconut Creek, FL 33073

New Seawall
Sheet 3 of 4

Design By: AWG Drawn By: AWG Checked By: AWG

RER - NATURAL RESOURCES DIVISION
FINAL APPROVAL
NAME [signature]
DATE 11/15/17

CCI-2017-0115

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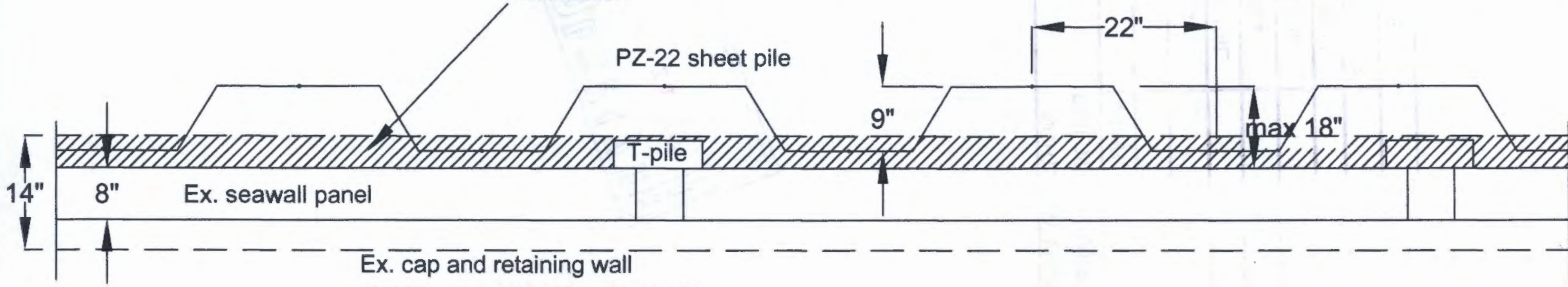

Placement of sheet pile panels within 18" of ex. seawall panel

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Coastal Resources Section
Resources Regulation & Restoration
Division (NRRRD)

ALAN W. GARCIA LICENSE No. 42564 STATE OF FLORIDA PROFESSIONAL ENGINEER

6/28/17

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4202 NW 54th Street
Coconut Creek, FL 33073
(954) 304-0098

Sheet Pile Placement
Sheet 1 of 1
Design By: AWG Drawn By: AWG Checked By: AWG

DERM
COPY

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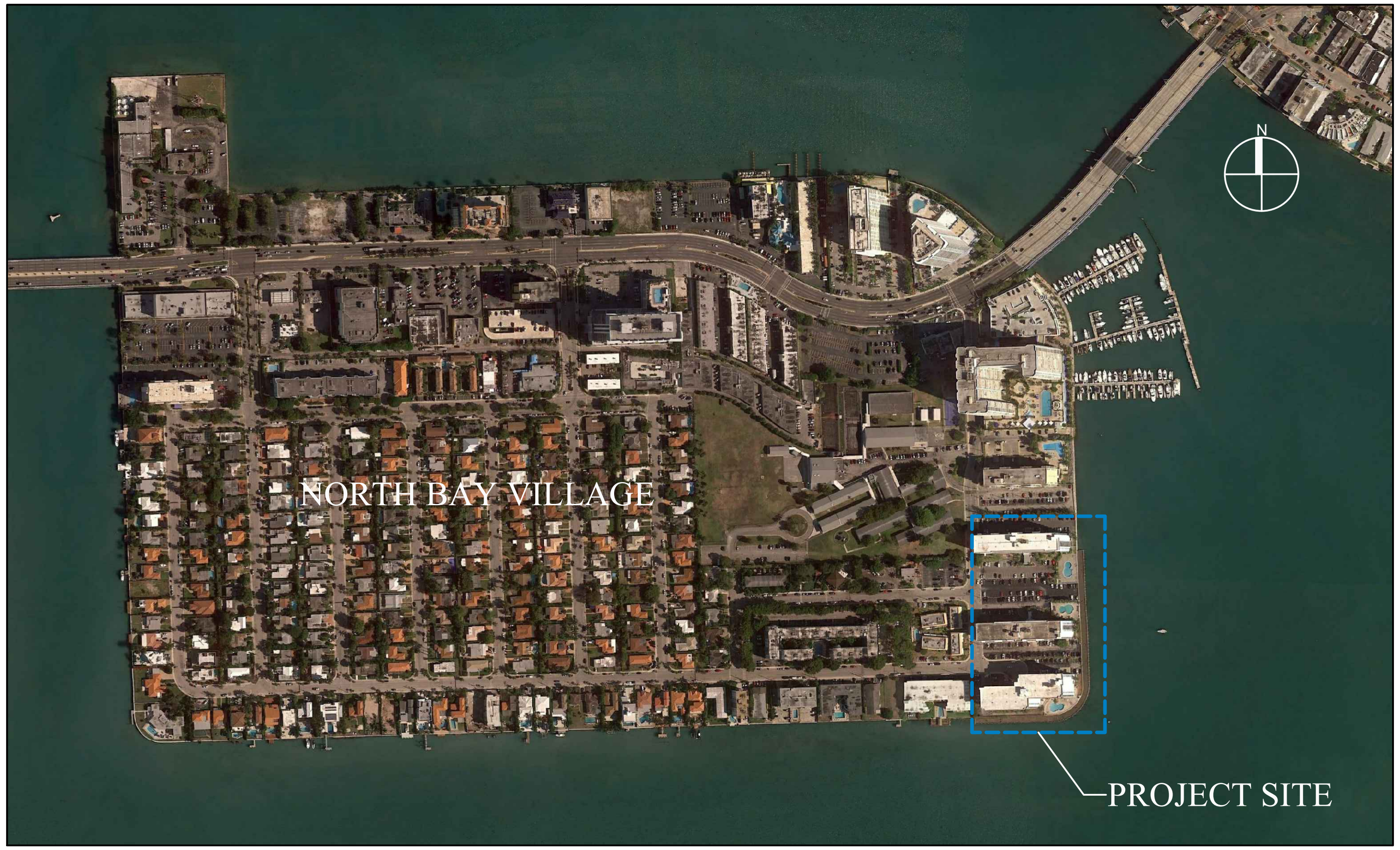
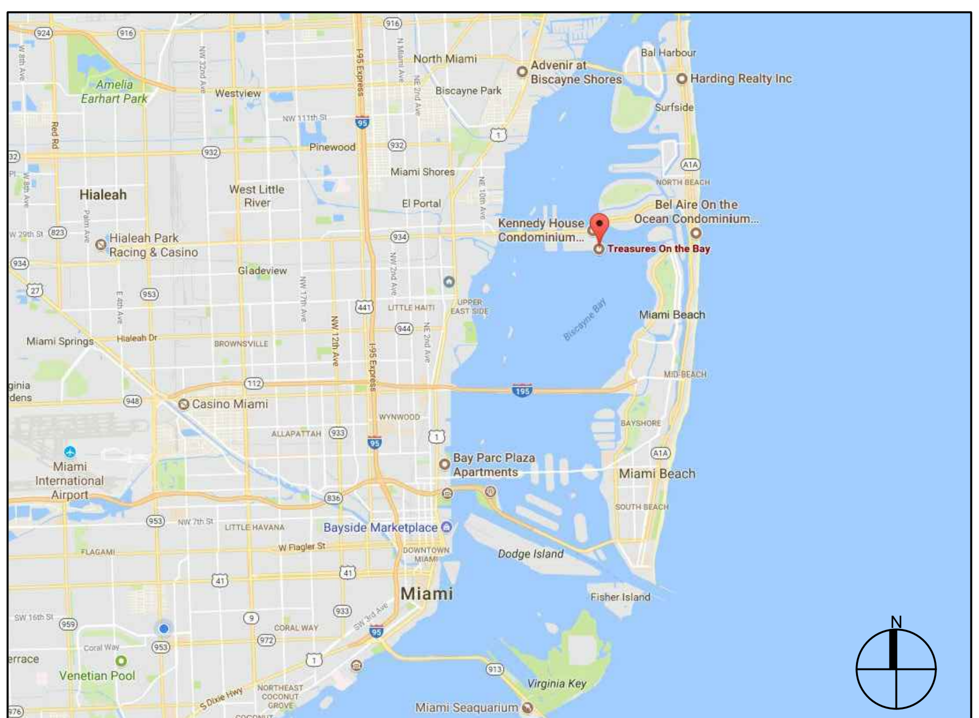
OCT 25 2017
By

NORTH BAY VILLAGE BUILDING DEPARTMENT				
DEPT	APPROVED	DENIED	DATE	INITIAL
ZONING				
BUILDING	OW		10-27-17	
STRUCTURAL	GVM		10-25-17	
PLUMBING				
ELECTRICAL				
MECHANICAL				
PUBLIC WORKS				
REMARKS:				



LOCATION MAP AND LEGAL DESCRIPTION



PROJECT SITE LOCATION:
1900 S Treasure Drive
North Bay Village, FL 33141

LATITUDE: 25°48'1.06"N
LONGITUDE: 80° 8'32.51"W

FOLIO No.: 23-3209-011-0120,

LEGAL DESCRIPTION:
PB 53-65
1ST ADD TO TREASURE ISLAND
LOTS 10 THRU 13 BLK 8 LESS
PORT DESC IN DECL OR 25577-2115
NAU 23 3209 055 K/A TREASURES
ON THE BAY III CONDO
LOT SIZE 32127 SQ FT
OR 22567-4551 0804 2
MASTER ASSOCIATION

GENERAL NOTES:

1. ELEVATIONS SHOWN REFER TO THE NATIONAL GEODETIC VERTICAL DATUM (NGVD) OF 1929.
2. ALL DIMENSIONS ON PLANS ARE SUBJECT TO VERIFICATION IN THE FIELD.
3. IT IS THE INTENT OF THESE PLANS TO BE IN ACCORDANCE WITH APPLICABLE CODES AND AUTHORITIES HAVING JURISDICTION. ANY DISCREPANCIES BETWEEN THESE PLANS AND APPLICABLE CODES SHALL BE IMMEDIATELY BROUGHT TO THE ATTENTION OF ENGINEER BEFORE PROCEEDING WITH WORK. CONTRACTOR AND ALL SUBCONTRACTORS ARE RESPONSIBLE FOR ALL LINES, ELEVATIONS, AND MEASUREMENTS IN CONNECTION WITH THEIR WORK.
4. IT IS THE INTENT OF THESE PLANS AND THE RESPONSIBILITY OF THE CONTRACTOR TO COMPLY WITH LOCAL, STATE, AND FEDERAL ENVIRONMENTAL PERMITS ISSUED FOR THIS PROJECT. IT SHALL BE THE CONTRACTOR'S RESPONSIBILITY TO FAMILIARIZE AND GOVERN HIMSELF BY ALL PROVISIONS OF THESE PERMITS.
5. APPLICABLE BUILDING CODE: FLORIDA BUILDING CODE, 2017 EDITION (AND CURRENT ADDENDUMS).
6. APPROVED CONTRACTOR TO DETERMINE THE SUITABILITY OF EXISTING STRUCTURES AND VERIFY ALL DIMENSIONS. THE APPROVED CONTRACTOR IS RESPONSIBLE FOR ALL METHODS, MEANS, SEQUENCES AND PROCEDURES OF WORK.
7. DO NOT SCALE DRAWINGS FOR DIMENSIONS.
8. CONTRACTOR TO VERIFY LOCATION OF EXISTING UTILITIES PRIOR TO COMMENCING WORK.
9. CONTRACTOR TO PROPERLY FENCE AND SECURE AREA WITH BARRICADES.
10. ANY DEVIATION AND/OR SUBSTITUTION FROM THE INFORMATION PROVIDED HEREIN SHALL BE SUBMITTED TO THE ENGINEER FOR APPROVAL PRIOR TO COMMENCEMENT OF WORK.
11. ALL NEW MATERIALS AND/OR PATCHWORK SHALL BE PROVIDED TO MATCH EXISTING MATERIALS AND/OR ADJOINING WORK WHERE PRACTICAL EXCEPT AS SPECIFICALLY NOTED HEREIN.
12. LICENSED CONTRACTOR SHALL USE ALL POSSIBLE CARE TO PROTECT ALL EXISTING MATERIALS, SURFACES, AND FURNISHINGS FROM DAMAGE DURING ALL PHASES OF CONSTRUCTION.
13. THE LICENSED CONTRACTOR TO INSTALL AND REMOVE ALL SHORING AND BRACING AS REQUIRED FOR THE PROPER EXECUTION OF THE WORK.
14. ALL NEW WORK AND/OR MATERIALS SHALL CONFORM TO ALL REQUIREMENTS OF EACH ADMINISTRATIVE BODY HAVING JURISDICTION IN EACH PERTAINING CIRCUMSTANCE.
15. THE CONTRACTOR WILL EMPLOY AND MAINTAIN ADEQUATE SEDIMENT AND EROSION CONTROL MEASURES TO PROTECT BISCAYNE BAY FROM SEDIMENT AND CONSTRUCTION DEBRIS.

DOCK: LL 100 PSF

BOLTS

1. ALL BOLTS SHALL BE STAINLESS STEEL, UNLESS OTHERWISE NOTED.

WOOD

1. PRIMARY WOOD FRAMING MEMBERS SHALL BE NUMBER 1 MARINE GRADE PRESSURE TREATED SOUTHERN PINE OR BETTER.
2. ALL DIMENSIONS ON PLANS ARE SUBJECT TO VERIFICATION IN THE FIELD.

CONCRETE

1. CONCRETE SHALL CONFORM TO ACI 318 (LATEST ED.) AND SHALL BE REGULAR WEIGHT, SULFATE RESISTANT, WITH A DESIGN STRENGTH OF 5,000 PSI AT 28 DAYS W/ A MAX WATER-CEMENTIOUS MATERIALS RATIO, BY WEIGHT, NORMAL WEIGHT AGGREGATE CONCRETE OF 0.40.
2. OWNER SHALL EMPLOY AND PAY FOR TESTING SERVICES FROM AN INDEPENDENT TESTING LABORATORY FOR CONCRETE SAMPLING AND TESTING IN ACCORDANCE W/ ASTM.
3. LICENSED CONTRACTOR IS RESPONSIBLE FOR THE ADEQUACY OF FORMS AND SHORING AND FOR SAFE PRACTICE IN THEIR USE AND REMOVAL.
4. CONCRETE COVER SHALL BE 3" UNLESS OTHERWISE NOTED ON APPROVED DRAWINGS.
5. REINFORCING STEEL SHALL BE IN CONFORMANCE WITH THE LATEST VERSION OF ASTM A615 GRADE 60 SPECIFICATIONS. ALL REINFORCEMENT SHALL BE PLACED IN ACCORDANCE W/ ACI 315 AND ACI MANUAL OF STANDARD PRACTICE.
6. SPLICES IN REINFORCING BARS SHALL NOT BE LESS THAN 48 BAR DIAMETERS AND REINFORCING SHALL BE CONTINUOUS AROUND ALL CORNERS AND CHANGES IN DIRECTION. CONTINUITY SHALL BE PROVIDED AT CORNERS OR CHANGES IN DIRECTION BY BENDING THE LONGITUDINAL STEEL AROUND THE CORNER 48 BAR DIAMETERS.

PILE DRIVING NOTES

1. PILE DRIVING OPERATIONS SHALL BE OBSERVED BY A SPECIAL INSPECTOR, INCLUDING TEST PILES SUFFICIENT TO DETERMINE THE APPROXIMATE LENGTH REQUIRED TO MEET DESIGN CAPACITY.
2. PILES SHALL BE DRIVEN USING AN APPROVED CUSHION BLOCK CONSISTING OF MATERIAL SO ARRANGED SO AS TO PROVIDE THE TRANSMISSION OF THE HAMMER ENERGY.
3. PILES SHALL BE DRIVEN TO REQUIRED CAPACITY A MIN 12. INTO BERM.
4. PILES SHALL BE DRIVEN WITH A DROP HAMMER OR GRAVITY HAMMER PROVIDED THE HAMMER SHALL WEIGH NO LESS THAN 3,000 POUNDS, AND THE FALL OF THE HAMMER SHALL NOT EXCEED 6 FT.
5. PILES SHALL BE DRIVEN WITH A VARIATION OF NOT MORE THAN $\frac{1}{4}$" PER FOOT FROM THE VERTICAL, OR FROM THE BATTER LINE INDICATED, WITH A MAX VARIATION OF THE HEAD OF THE PILE FROM THE POSITION SHOWN ON THE PLANS OF NOT MORE THAN 3".
6. WHERE PILING MUST PENETRATE STRATA OFFERING HIGH RESISTANCE TO DRIVING, THE STRUCTURAL ENGINEER OF RECORD OR SPECIAL INSPECTOR MAY REQUIRE THAT PILES BE SET IN PRE-DRILLED OR PUNCHED HOLES. THE PILES SHALL REACH THEIR FINAL PENETRATION BY DRIVING.

TREASURES ON THE BAY WATERFRONT IMPROVEMENT PROJECT

CLIENT:
TREASURES ON THE BAY MASTER ASSOCIATION, INC.
c/o **MR. ALBERTO PARJUS**
1900 South Treasure Drive
North Bay Village, FL 33141

ENVIRONMENTAL CONSULTANT:
OCEAN CONSULTING, LLC
340 Minorca Avenue, Suite 7
Coral Gables, Florida 33134
Tel: (305) 921-9344
Fax: (305) 677-3254

CONTRACTOR:

PROJECT ENGINEER:
DYNAMIC ENGINEERING SOLUTIONS, INC.
351 S. Cypress Road, Suite 303
Pompano Beach, FL 33060
Office - 954-545-1740
Fax - 954-545-1721

SEAL / SIGNATURE / DATE

John Omslaer
PE 52733, CA 26829

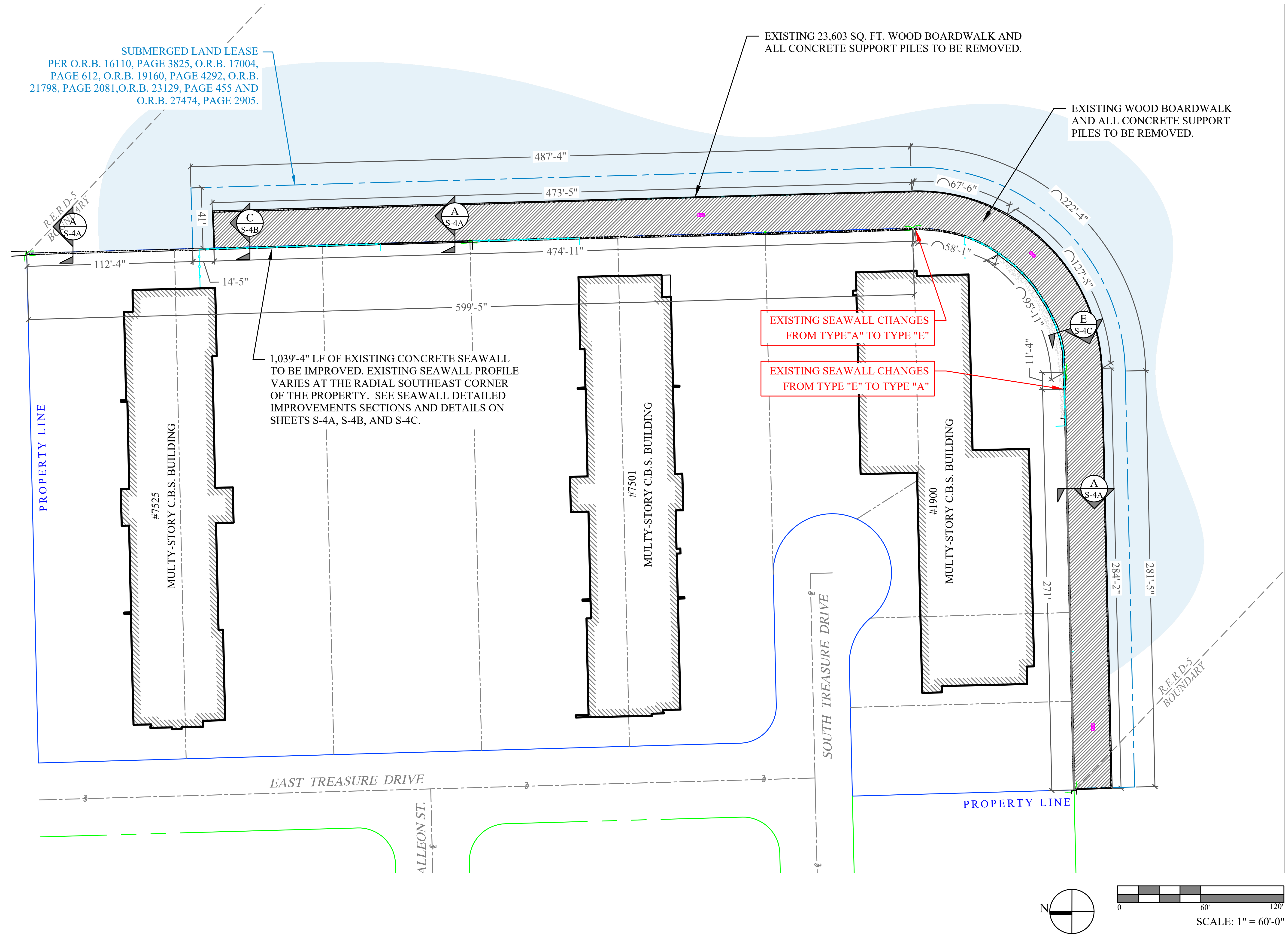
PERMIT DRAWINGS

Issue #	**Issue Date**
1	September 25, 2018

PROJECT: 17-6630

PROJECT LOCATION & NOTES

SCALE : AS SHOWN
SHEET NO.
S-1



TREASURES ON THE BAY WATERFRONT IMPROVEMENT PROJECT

CLIENT:
TREASURES ON THE BAY MASTER ASSOCIATION, INC.
c/o **MR. ALBERTO PARJUS**
1900 South Treasure Drive
North Bay Village, FL 33141

ENVIRONMENTAL CONSULTANT:
OCEAN CONSULTING, LLC
340 Minorca Avenue, Suite 7
Coral Gables, Florida 33134
Tel: (305) 921-9344
Fax: (305) 677-3254

CONTRACTOR:

PROJECT ENGINEER:
DYNAMIC ENGINEERING SOLUTIONS, INC.
351 S. Cypress Road, Suite 303
Pompano Beach, FL 33060
Office - 954-545-1740
Fax - 954-545-1721

SEAL / SIGNATURE / DATE

John Omslaer
PE 52733, CA 26829

PERMIT DRAWINGS

Issue #	Issue Date
1	September 25, 2018

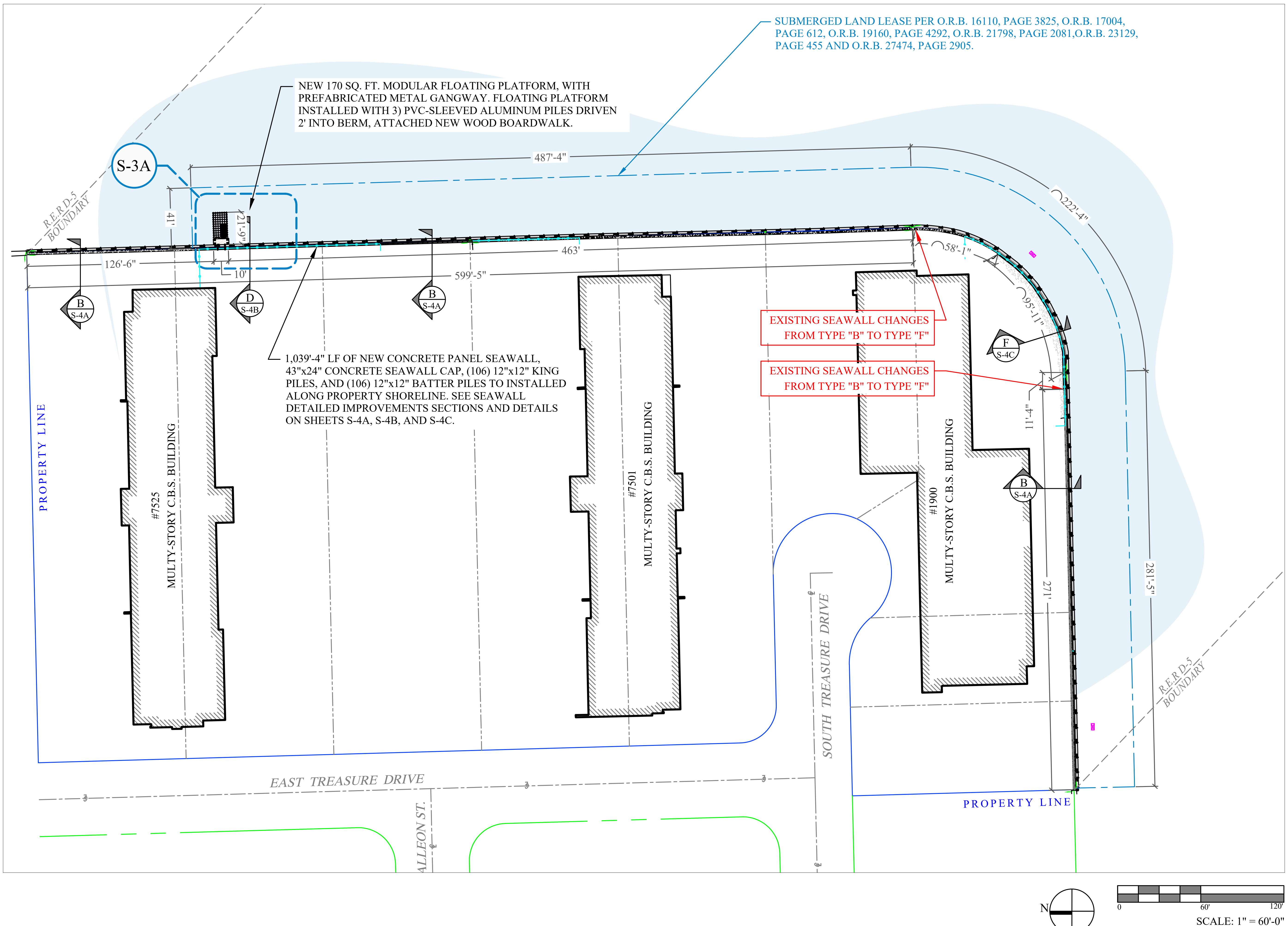
PROJECT: 17-6630

EXISTING SITE PLAN CONDITIONS

SCALE : AS SHOWN

SHEET NO.

S-2



TREASURES ON THE BAY WATERFRONT IMPROVEMENT PROJECT

CLIENT:
TREASURES ON THE BAY MASTER ASSOCIATION, INC.
c/o **MR. ALBERTO PARJUS**
1900 South Treasure Drive
North Bay Village, FL 33141

ENVIRONMENTAL CONSULTANT:
OCEAN CONSULTING, LLC
340 Minorca Avenue, Suite 7
Coral Gables, Florida 33134
Tel: (305) 921-9344
Fax: (305) 677-3254

CONTRACTOR:

PROJECT ENGINEER:
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351 S. Cypress Road, Suite 303
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Fax - 954-545-1721

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1	September 25, 2018

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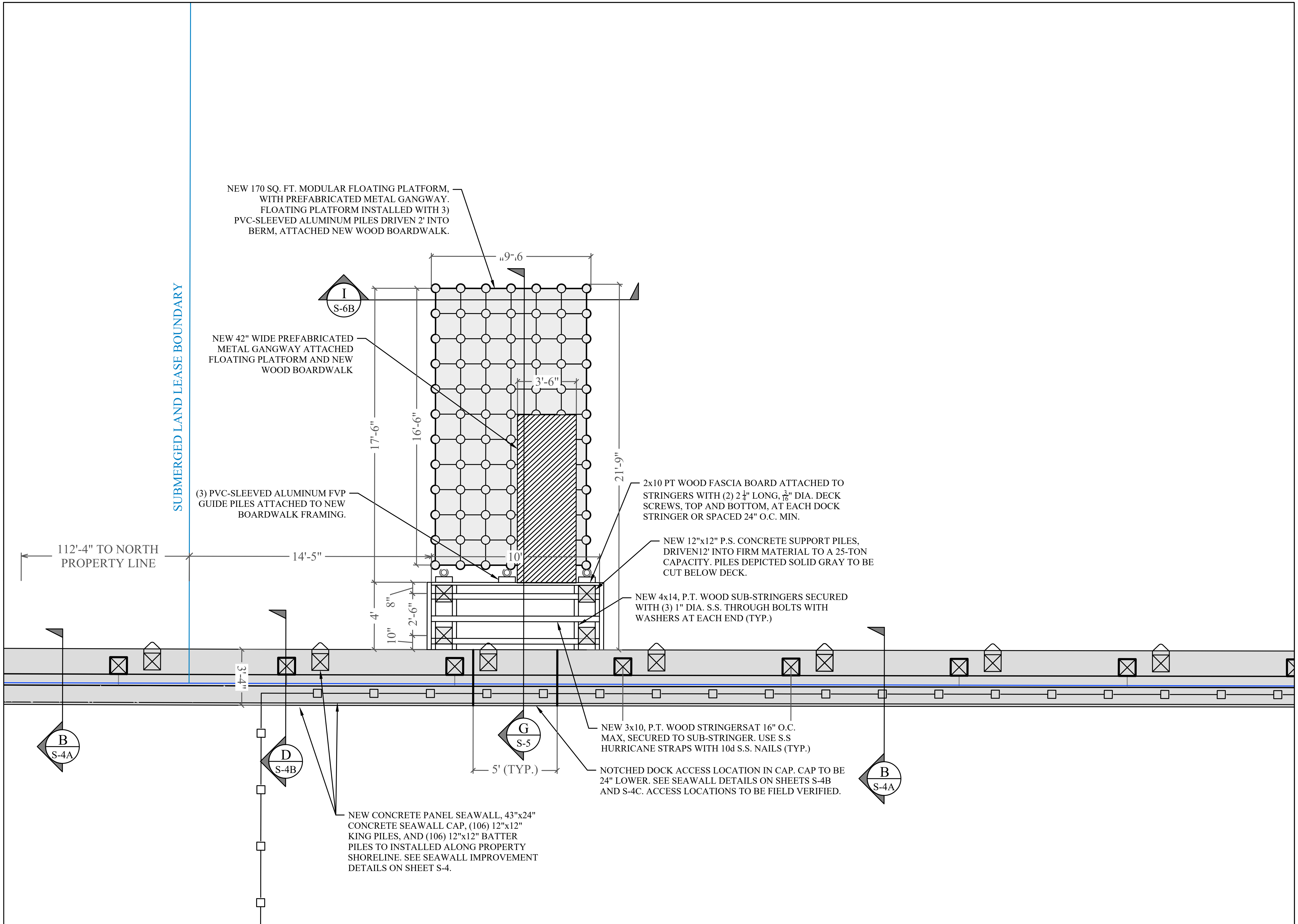
PROPOSED SITE PLAN CONDITIONS

SCALE : AS SHOWN

SHEET NO.

S-3



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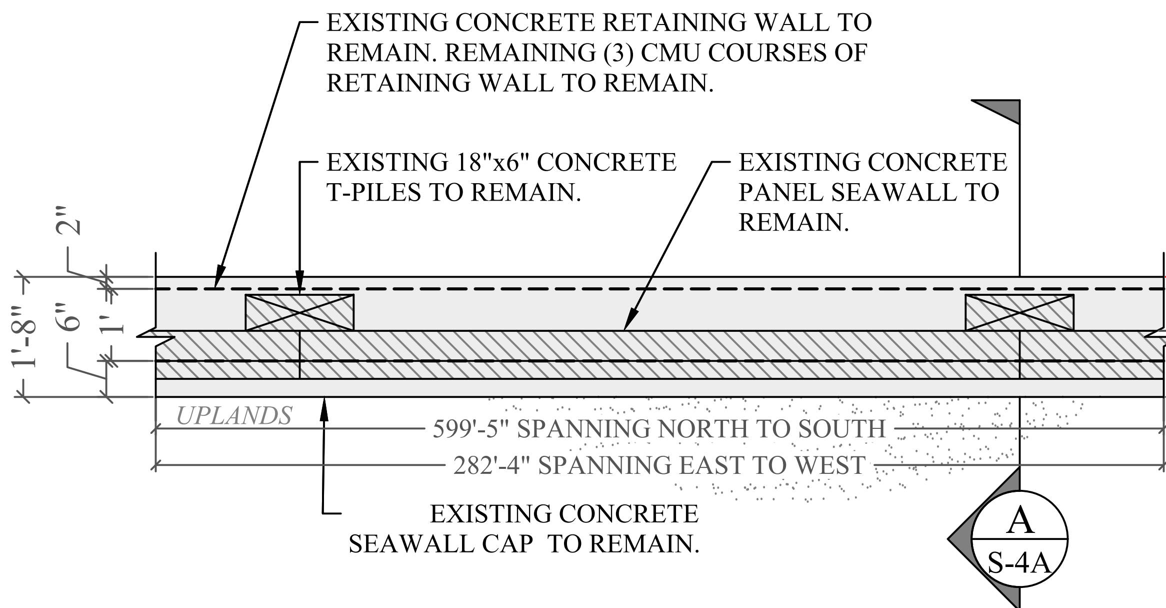
TYPICAL BOARDWALK FRAMING PLAN

SCALE : AS SHOWN

SHEET NO.

S-3A



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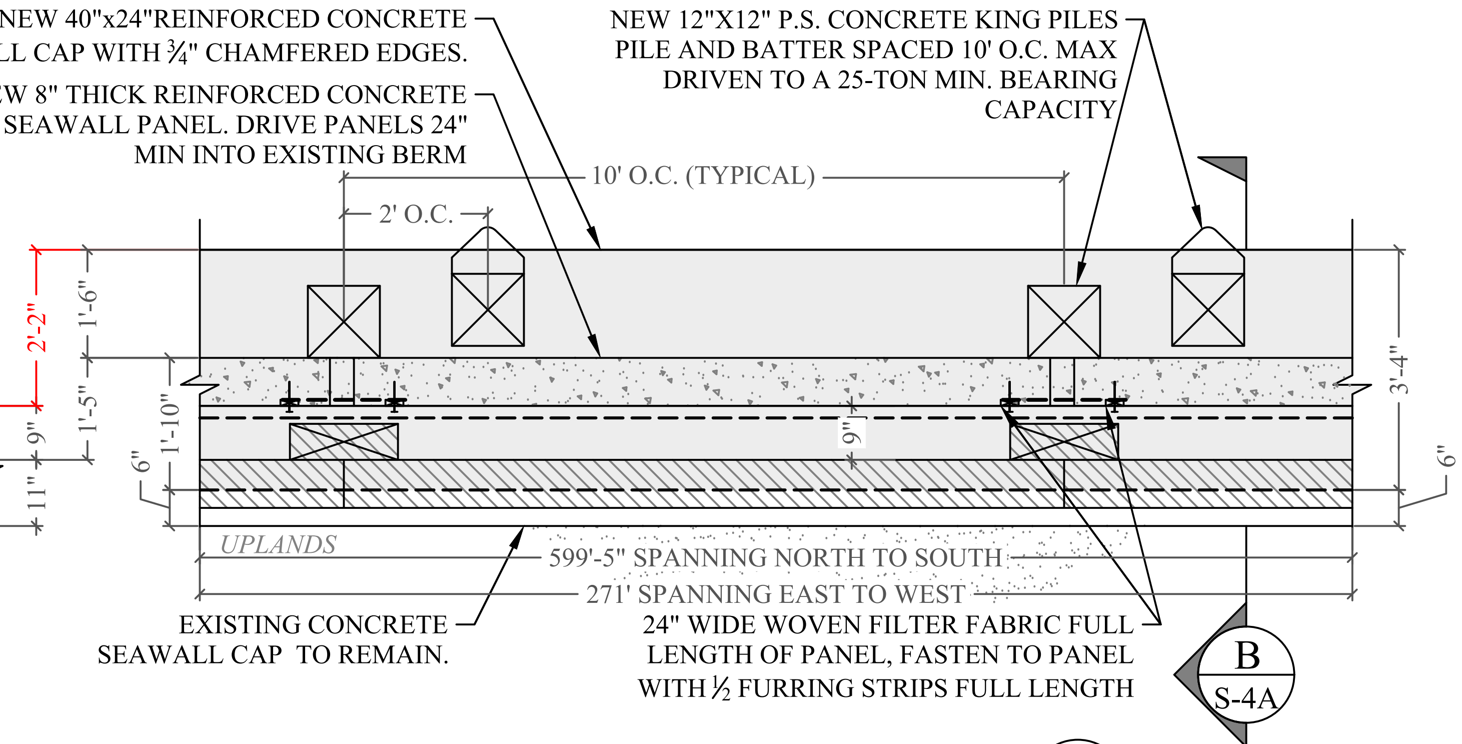

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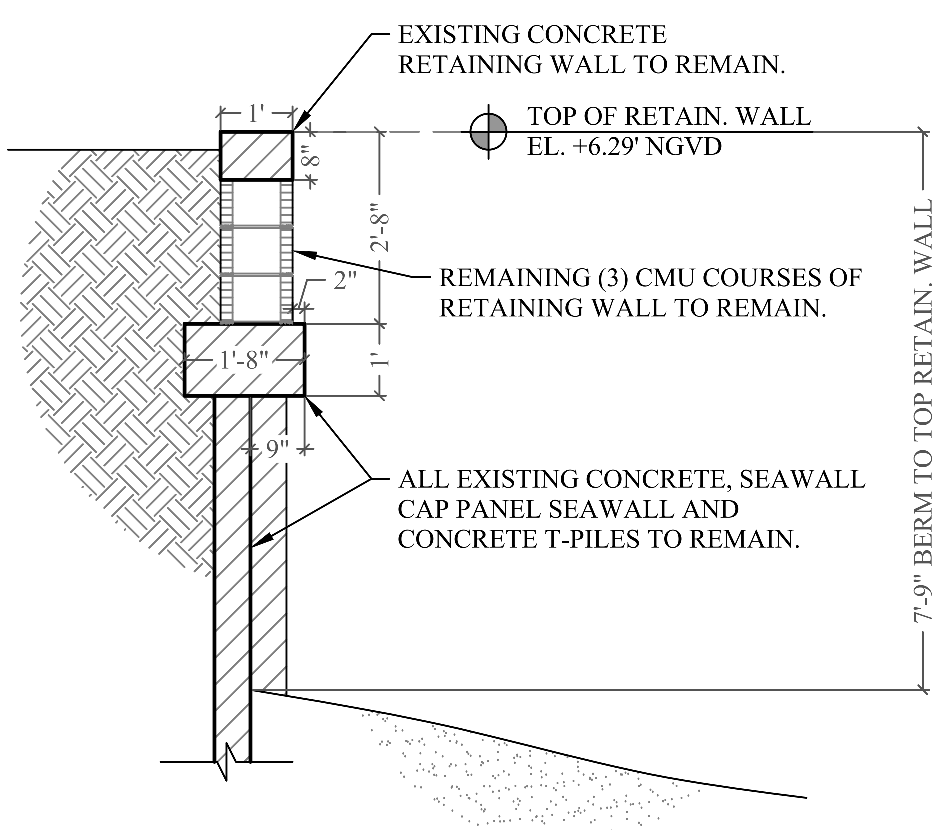

TYP. EXISTING SEAWALL SECTION A S-4A

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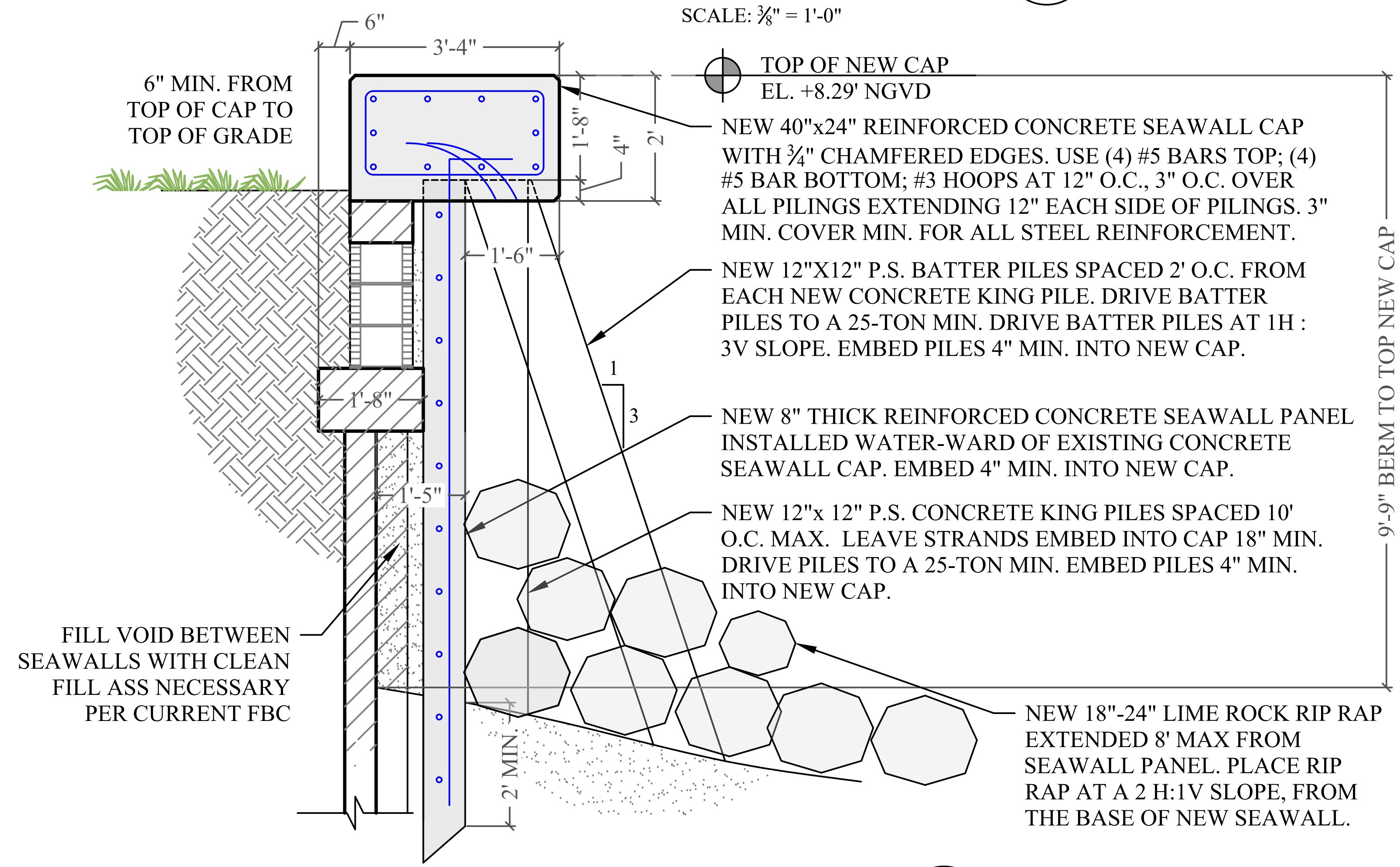

TYP. PROPOSED SEAWALL SECTION B S-4A

SCALE: 3/8" = 1'-0"

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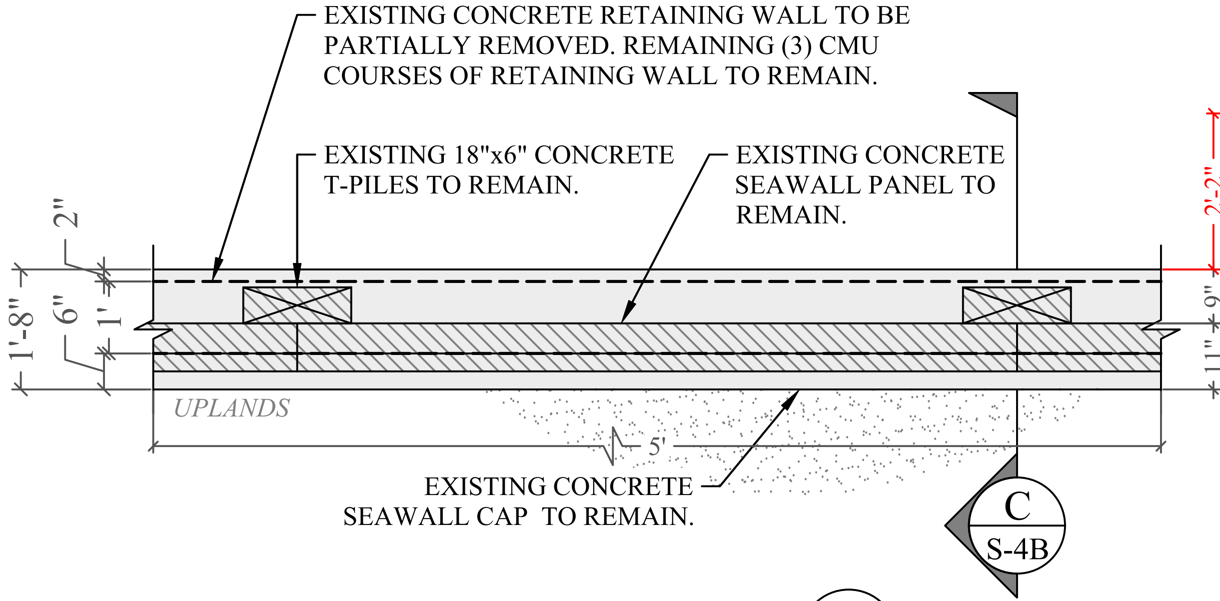
TYPICAL SEAWALL DETAILS & SECTIONS

SCALE : AS SHOWN

SHEET NO.

S-4A



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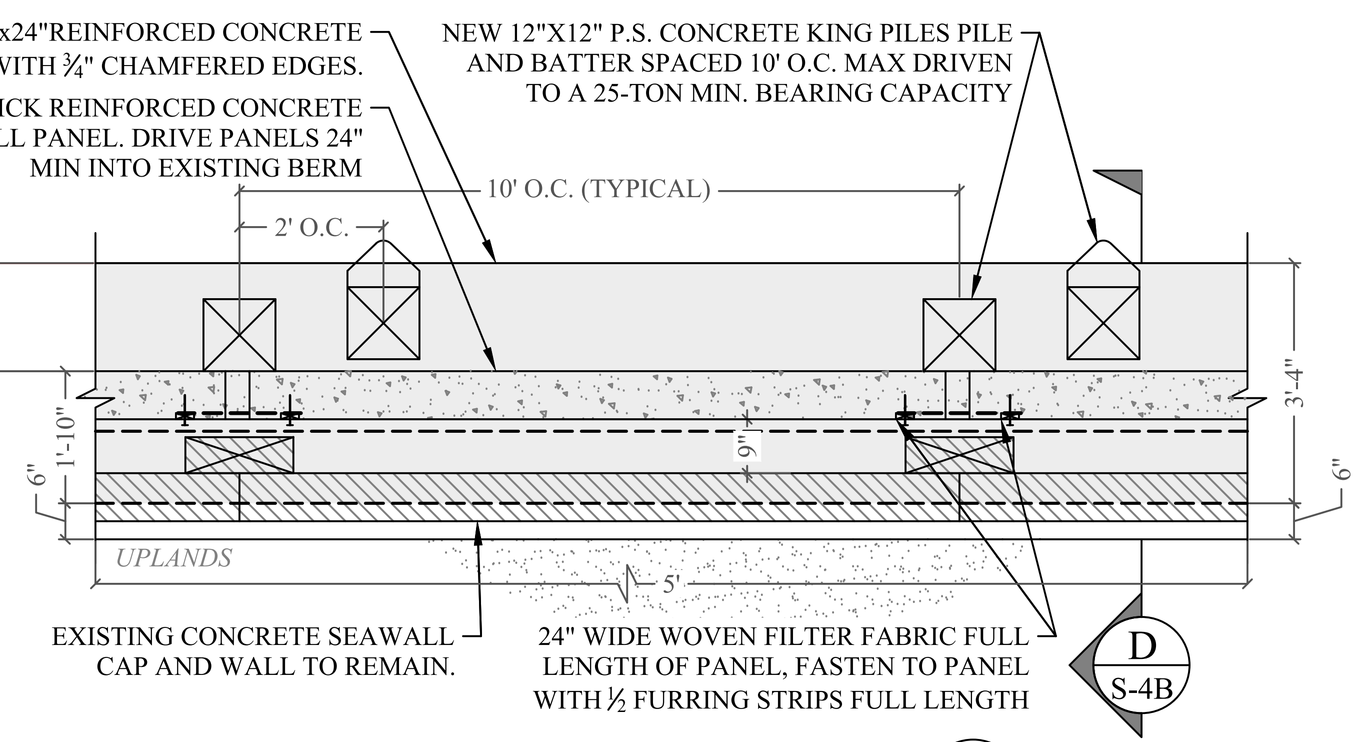

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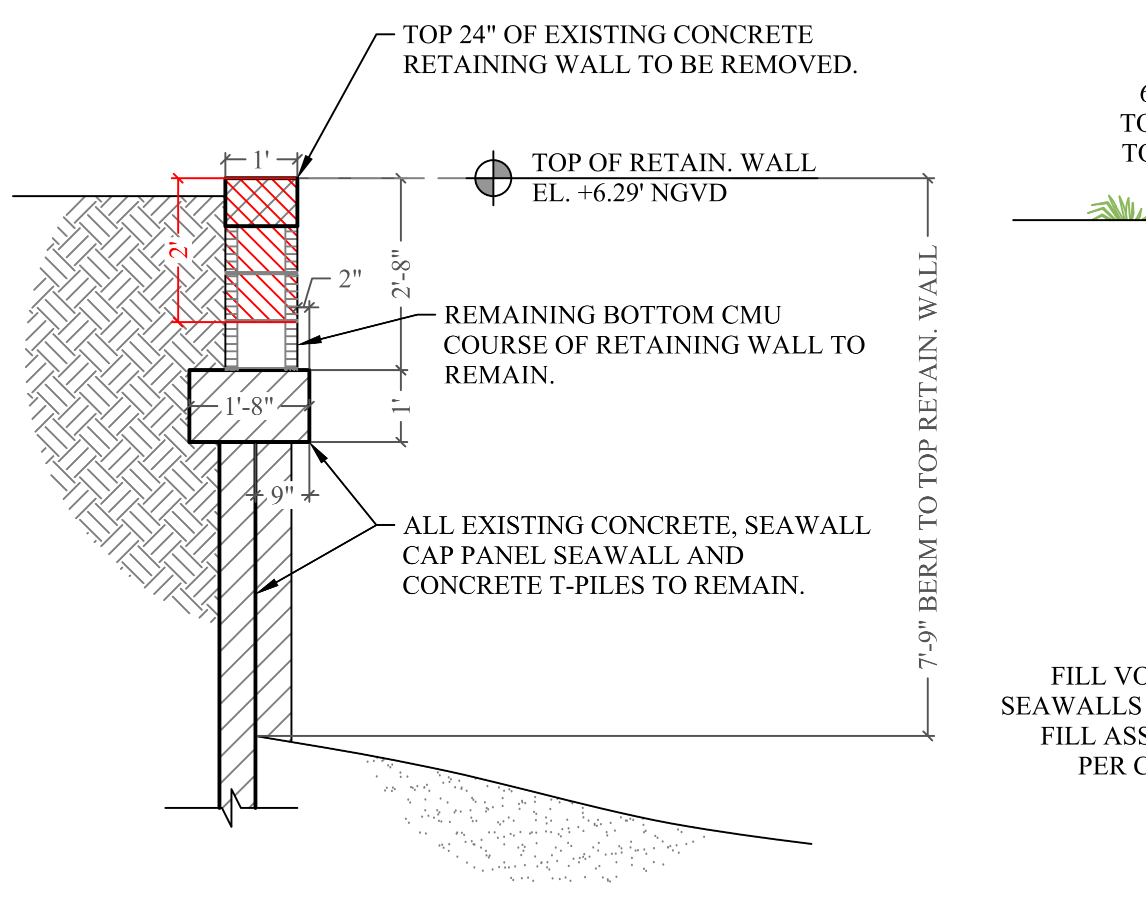

TYP. EXISTING SEAWALL SECTION C S-4B
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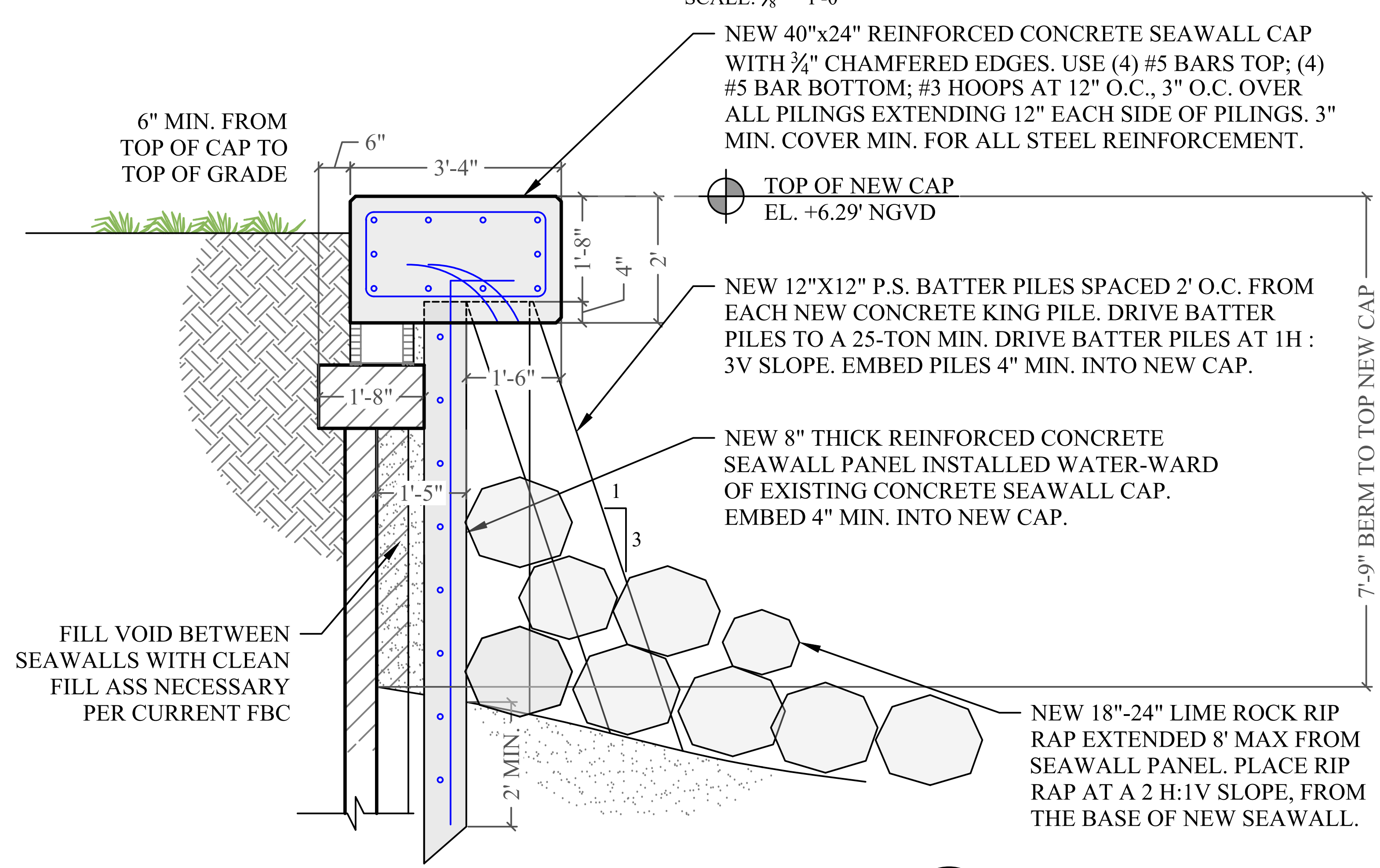

TYP. PROPOSED SEAWALL SECTION D S-4B
SCALE: 3/8" = 1'-0"

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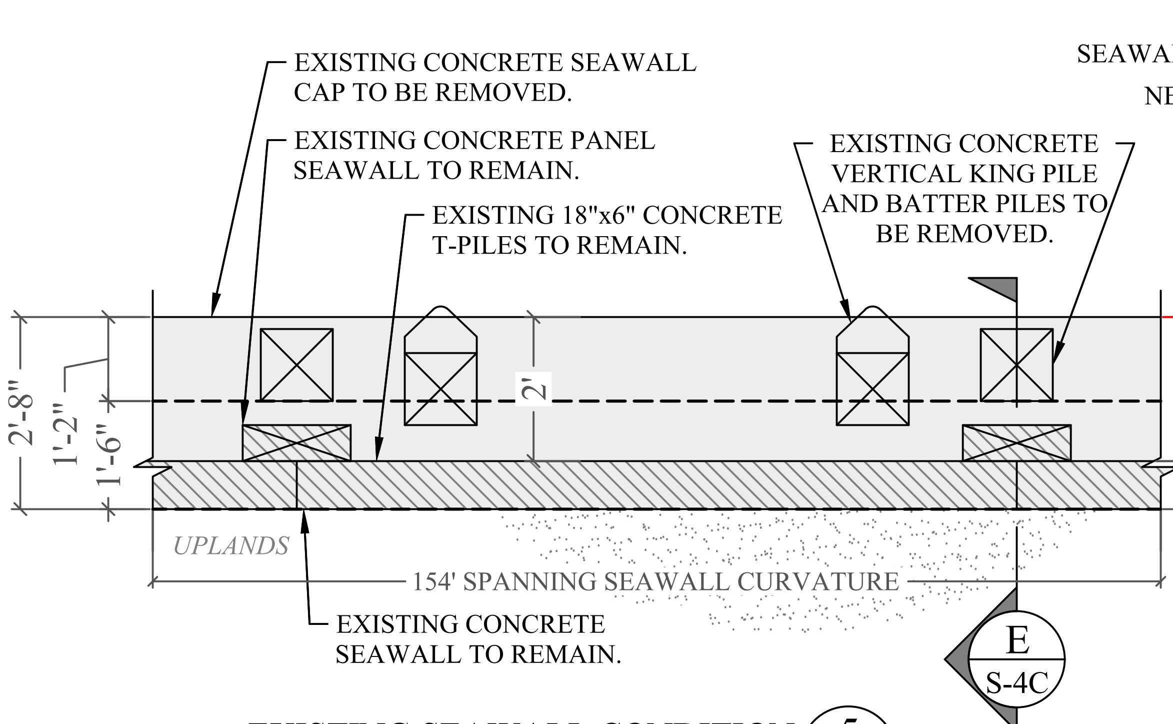
DOCK ACCESS TYPICAL SEAWALL DETAILS & SECTIONS

SCALE : AS SHOWN

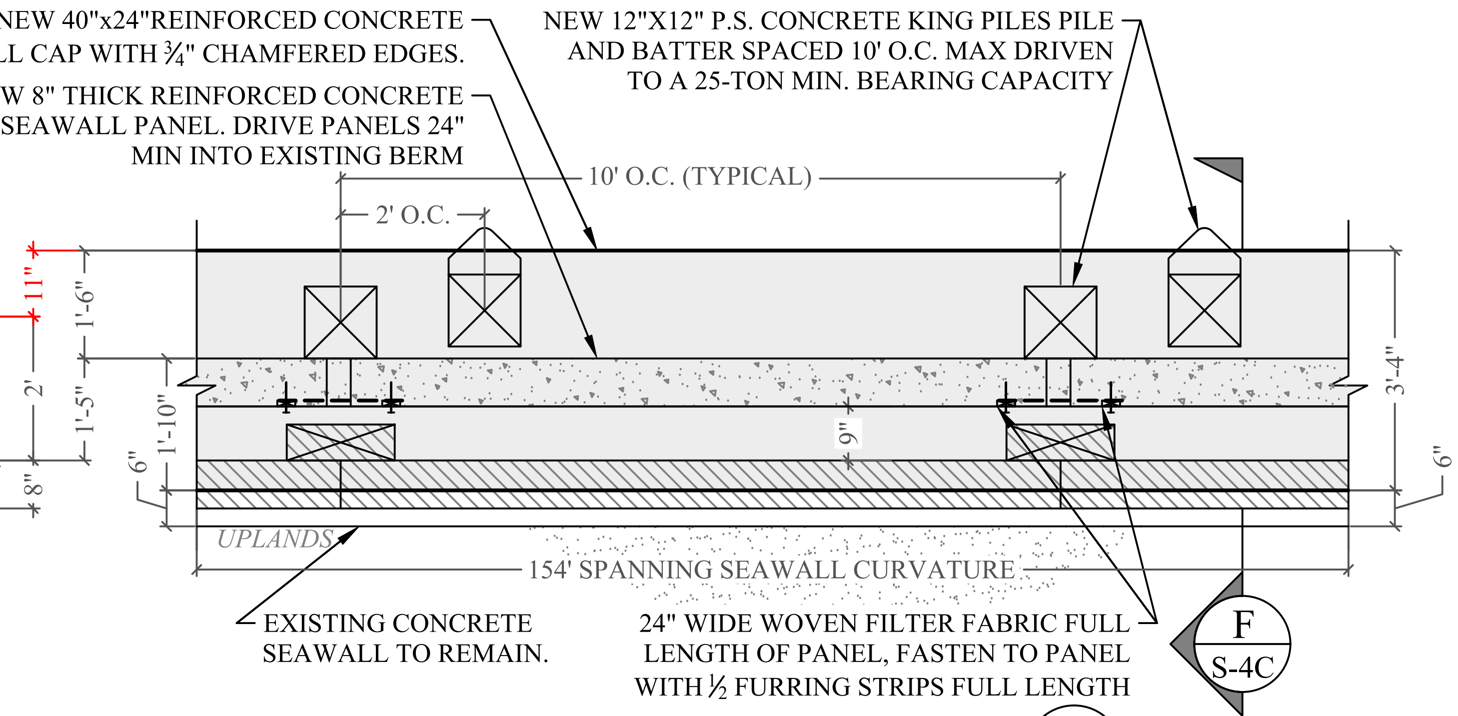
SHEET NO.

S-4B



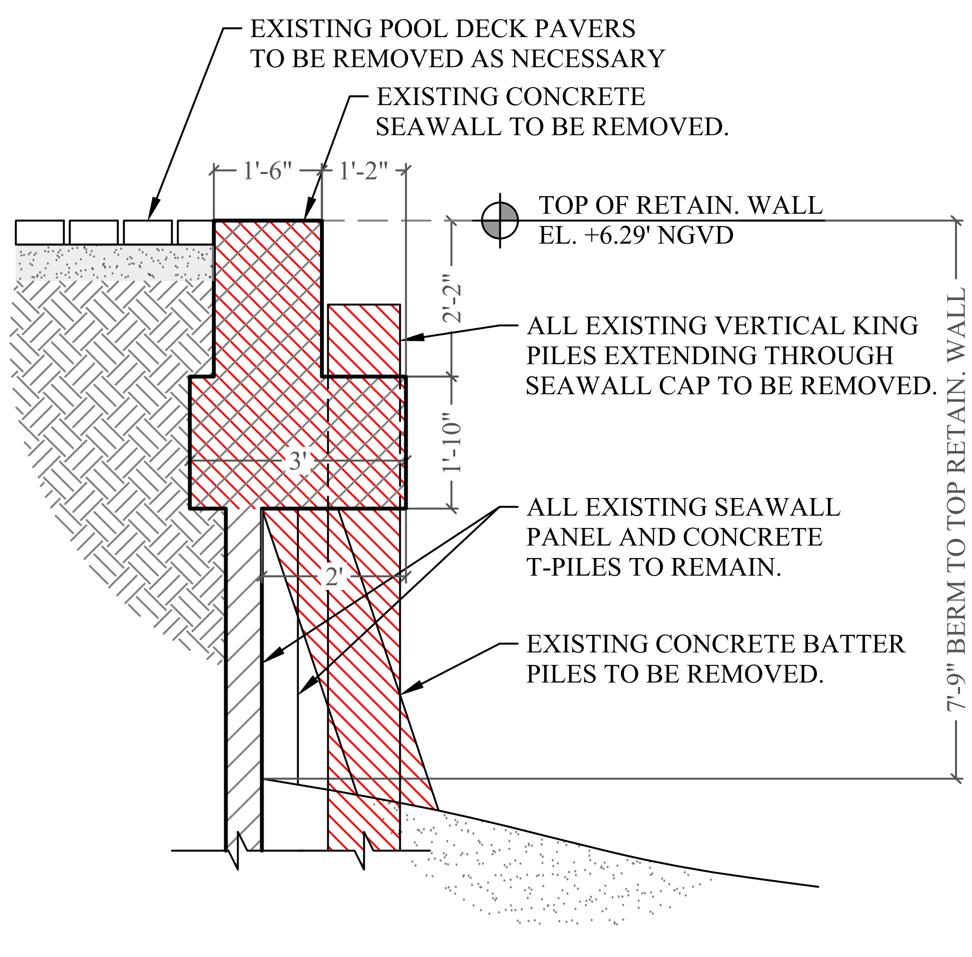
EXISTING SEAWALL CONDITION DETAIL (TYP.) 5 S-4C

SCALE: 3/8" = 1'-0"



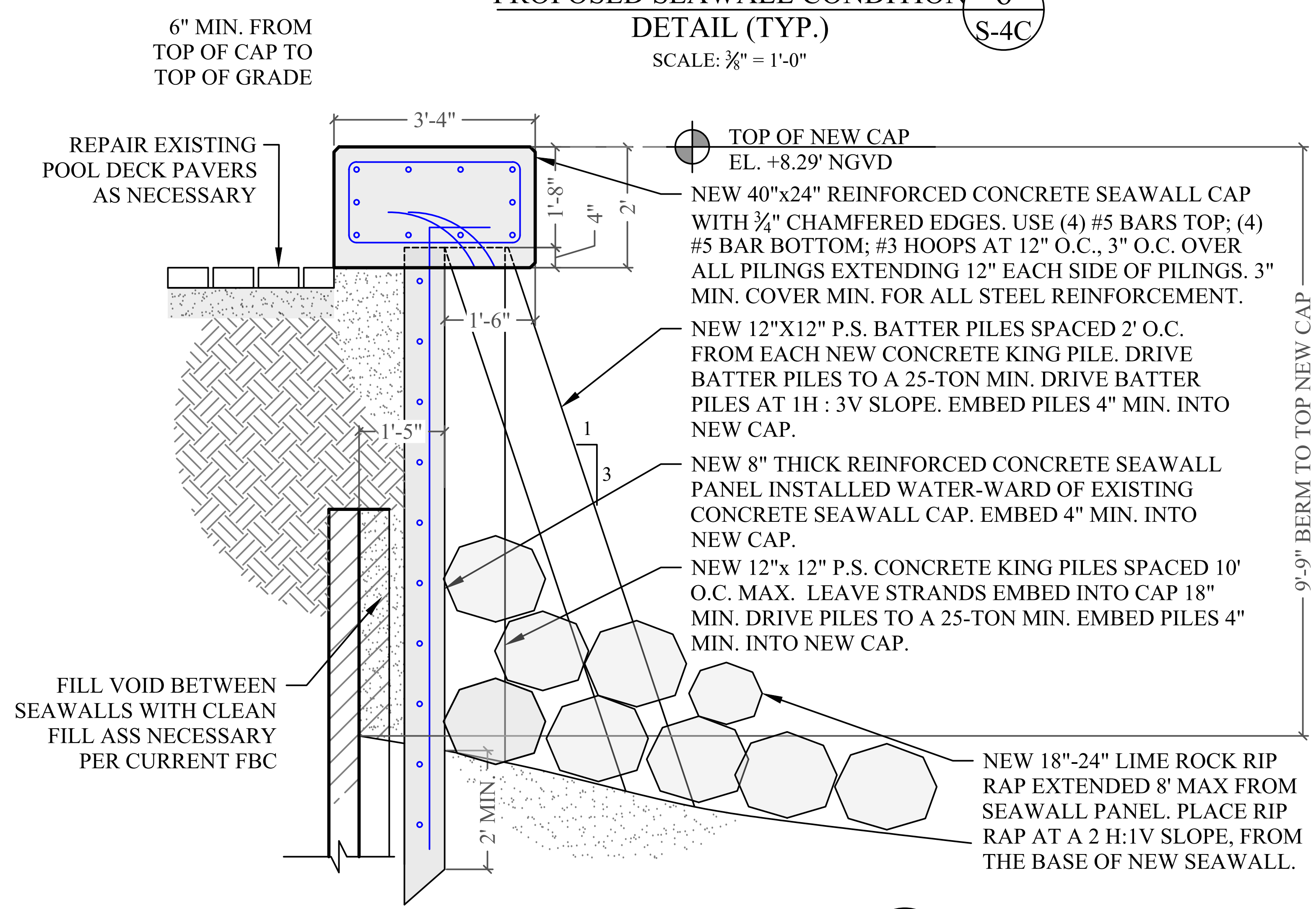
PROPOSED SEAWALL CONDITION DETAIL (TYP.) 6 S-4C

SCALE: 3/8" = 1'-0"



TYP. EXISTING SEAWALL SECTION E S-4C

SCALE: 3/8" = 1'-0"



TYP. PROPOSED SEAWALL SECTION F S-4C

SCALE: 3/8" = 1'-0"

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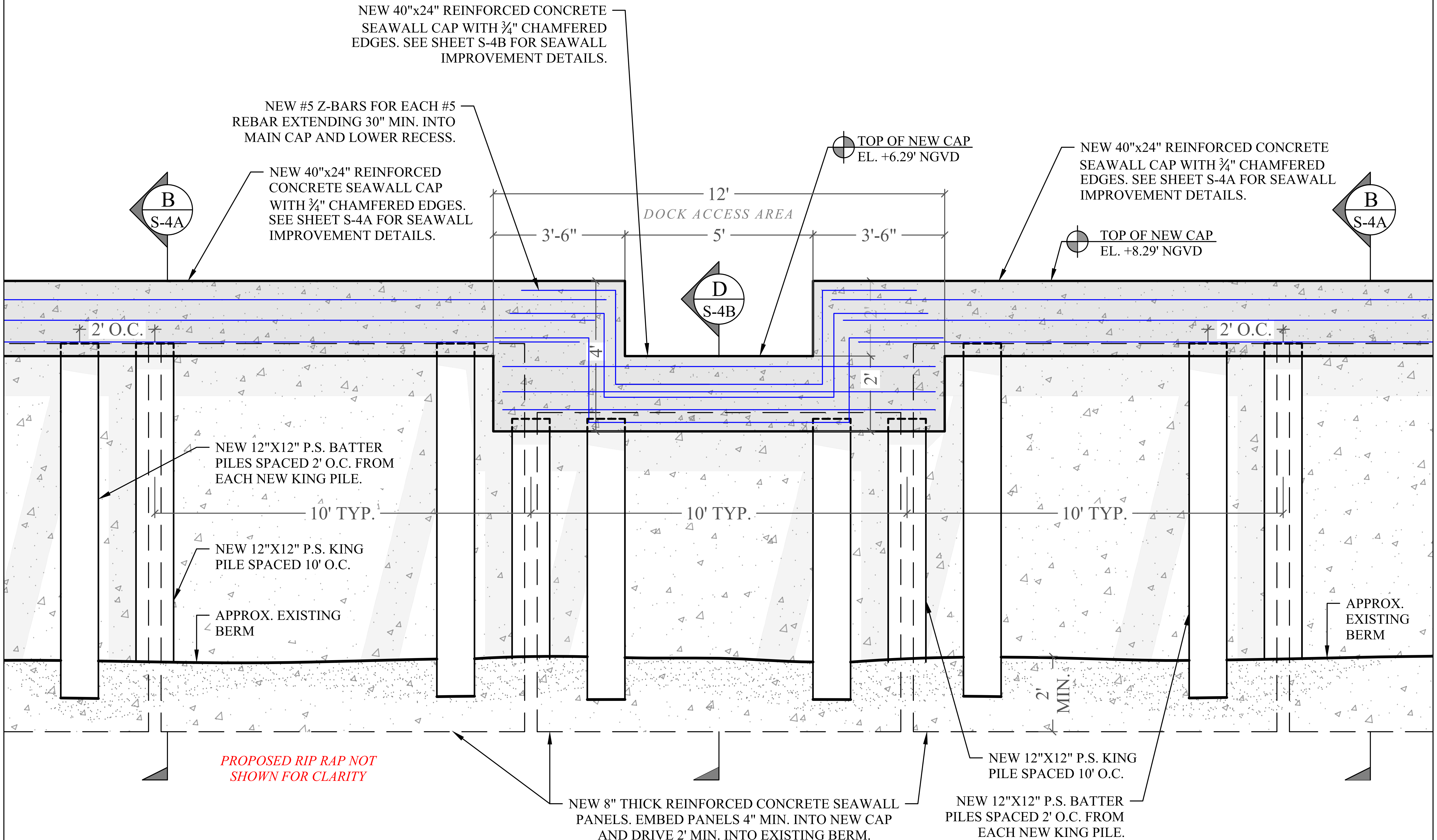
DOCK ACCESS TYPICAL SEAWALL DETAILS & SECTIONS

SCALE : AS SHOWN

SHEET NO.

S-4C



DOCK ACCESS CAP TRANSITION ELEVATION DETAIL 7 / S-4D

SCALE: ⅜" = 1'-0"

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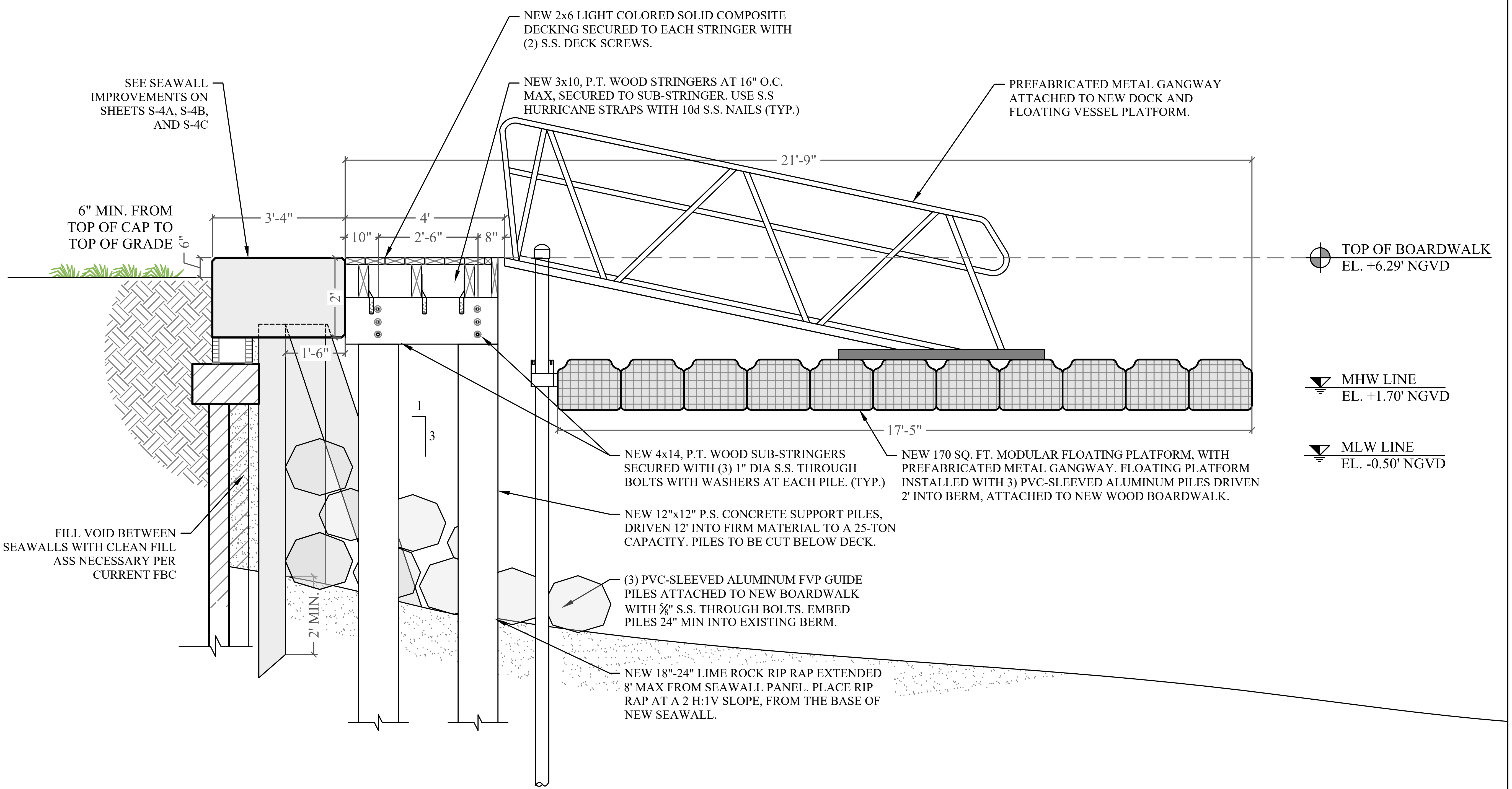
TYP. SEAWALL CAP TRANSITION ELEVATION DETAIL

SCALE : AS SHOWN

SHEET NO.

S-4D



FLOATING PLATFORM GANGWAY SECTION (G / S-5)

SCALE: 3/8" = 1'-0"

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1	September 25, 2018

PROJECT: 17-6630

FLOATING VESSEL PLATFORM SECTION

SCALE : AS SHOWN

SHEET NO.

S-5

Appendix B

GCES Geotechnical Report

EAC CONSULTING, INC

North Bay Village Seawalls
North Bay Village
Miami-Dade County, FL
GCES Project No. G10201005

Geotechnical Engineering Services

Report of Subsurface Exploration

GCES ENGINEERING SERVICES, LLC
10860 NW 138th Street, Unit 4
Hialeah Gardens, FL 33018
305.964.0669 I C: 954.440.8623
www.gces-usa.com

March 17, 2020

GCES Engineering Services, LLC.
10860 NW 138th Street I Unit 4 I Hialeah Gardens, FL 33018
P: 305.964.0669 I C: 954.440.8623
www.gces-usa.com

March 17, 2020

Evelyn Rodriguez, P.E., ENV SP
Assistant Project Manager
EAC Consulting, Inc.
5959 Blue Lagoon Drive, Suite 410
Miami, FL 33126

Subject: Geotechnical Engineering Services
Report of Subsurface Exploration
North Bay Village Seawalls
North Bay Village, Miami-Dade County, FL
GCES Project Number G10201005

Dear Evelyn:

GCES Engineering Services, LLC (GCES) has completed the Geotechnical Report for the North Bay Village Seawalls for the above referenced project. This work was performed as authorized in our agreement with EAC Consulting, Inc.

The report presents the results of our field exploration and laboratory testing programs along with our geotechnical data for use in design of the proposed seawalls.

We appreciate the opportunity to provide our services on this project. If you have any questions concerning the information provided, please do not hesitate to contact our office.

GCES ENGINEERING SERVICES, LLC

Dhayana Chacon
Engineering Staff

Alejandro R. Montenegro, P.E.
Senior Geotechnical Engineer
Florida PE # 59426

ALEJANDRO R. MONTENEGRO LICENSE No. 59426 STATE OF FLORIDA PROFESSIONAL ENGINEER

3/17/20

C:\G10201005 - North Bay Village Seawalls

TABLE OF CONTENTS

APPENDICES

1.0 INTRODUCTION

1.1 General

GCES Engineering Services. LLC, (GCES) has completed the subsurface exploration for the proposed Seawalls in North Bay Village, Miami-Dade County, Florida.

The project consists of providing geotechnical recommendations in connection with the construction of new seawalls at three locations throughout the Village along their shoreline. This report describes the subsurface conditions encountered in the borings, analyzes and evaluates the field and laboratory test data, and provides geotechnical information for the design of the proposed seawalls.

1.2 Project Location

The subject sites are within the City of North Bay Village, Miami-Dade County, FL. The sites for the new seawalls are located near the intersections shown below:

- NE Quadrant of intersection of East Drive and John F. Kennedy Causeway
- NW quadrant of intersection of Bounty Avenue and South Treasure Drive
- NW Quadrant of Intersection John F. Kennedy Causeway and E. Treasure Drive

We have appended a project Vicinity Map, Figure 1, which identifies the location of each study area. This map is presented in Appendix A.

1.3 Project Description

GCES understands that the project consists of constructing new seawalls throughout the Village along their shoreline near the locations shown above. GCES was provided with a Site Plan sent via email on February 14, 2020 provided by Ms. Abbie Wilson, P.E. of Moffatt & Nichol. The site plan shows the approximate location of the soil borings. GCES was also provided with a map of boundary and topographic survey prepared by Hadonne dated August 29, 2014. This map shows the existing conditions and ground surface elevations within the proposed seawall located the NW Quadrant of Intersection John F. Kennedy Causeway and E. Treasure Drive. Ground surface elevates at the other two locations indicated above were not provided.

If any of our understandings is not correct or if the structure differs from the characterization we have provided in this report, please inform us immediately so that we may re-evaluate our analyses.

2.0 SCOPE OF SERVICES

Our services for this project consisted of providing the following geotechnical engineering services:

- Conducted a field reconnaissance prior to the subsurface exploration.
- Assessed conditions with respect to the drilling equipment access, general topographic site conditions, property restrictions, overhead utilities, and utility underground.
- Marked the boring locations in the field by GCES personnel using layout procedures.
- Coordinated with utility locating service to locate utilities within rights-of-ways and easements for the borings.
- Reviewed of available subsurface test data, such as the "Soil Survey of Miami-Dade County, Florida" published by the United States Department of Agriculture (USDA).
- Performed a total of three (3) Standard Penetration Test (SPT) borings to depths of about 35 feet deep below existing grades in the vicinity of proposed retaining wall structures.
- Visually classified soil samples using the Unified Soil Classification System (USCS) and performed laboratory tests on selected representative samples to evaluate the physical and engineering properties of the strata observed.
- Provide estimated soil design parameters (i.e. unit weights, angle of friction and earth pressure coefficients) for use in seawall design.

- Reviewed field and laboratory data, then prepared an engineering report summarizing our field and laboratory testing, subsurface soil and groundwater conditions for design of the proposed structures.

3.0 FIELD EXPLORATION AND LABORATORY TESTING

3.1 STANDARD PENETRATION TEST (SPT)

GCES's field exploration consisted of performing three (3) Standard Penetration Test (SPT) borings, B-1 through B-3 to depths of 35 feet below existing grades for the proposed seawalls. The field exploration was conducted on March 10, 2020.

The borings were performed in areas accessible to our drilling equipment and in areas that were not conflicting with existing underground utilities.

The SPT borings were performed using a truck-mounted drill rig equipped with a calibrated automatic hammer. The boreholes were advanced using drilling mud techniques and casing. The borings were performed in general accordance with ASTM Standard D-1586.

The SPT boring was continuously sampled in the upper 10 feet. Thereafter, the sampling interval was every 5 feet. Each boring was logged by the on-site personnel during the field exploration. Disturbed soil samples were placed in glass jars or sealed plastic bags and returned to our laboratory for additional visual classification by a GCES Engineer. Upon completion of the SPT borings, the boreholes were backfilled with cement grout, the surface restored (with cold mix asphalt where applicable), and the site cleaned as required.

The results of the SPT tests are presented on the boing logs included in Appendix B. The boring logs represent an interpretation of the field logs and includes modifications based on a geotechnical engineer's visual classification of the samples returned to the laboratory.

A brief description of the field exploration procedures employed in our subsurface investigation is provided in Appendix C of this report.

3.2 WATER LEVEL MEASUREMENTS

Water level depths were obtained during the test boring operations. In relatively previous soils, such as sandy (granular) soils, the indicated depths are usually groundwater levels. Seasonal variations, tidal conditions, temperature, land use, and recent rainfall conditions may influence the depths of the groundwater.

3.3 LABORATORY TESTING

Representative samples collected from the SPT borings were visually reviewed in the laboratory by a geotechnical engineer to confirm the field classifications. The descriptions of the soils indicated in the boring logs are in general accordance with the enclosed General Notes, Unified Soil Classification System (USCS), and American Society of Testing and materials (ASTM-2488).

The classification was based on visual observations, texture, and consistency with the aids of laboratory testing. The tests were performed on selected samples believed to be representative of the materials encountered. Designated group symbols according to the Unified Soil Classification System are given on the boring logs.

A brief description of the USCS classification system is attached to this report, Appendix C. A brief description of the laboratory testing procedure employed in our subsurface investigation is provided in Appendix C of this report.

4.0 SITE AND SUBSURFACE CONDITIONS

4.1 SITE CONDITIONS

Our understanding of the existing site conditions is based on the information provided to us by EAC Consulting, Moffatt & Nichol and our observations during the field exploration.

Our understanding of the site conditions is based on our initial field review and our observations during the performance of the field exploration program. The proposed seawalls will be constructed at the shorelines near the intersections indicated in section 1.2. The locations of the proposed seawalls are shown in Figure 1 presented in Appendix A. The areas adjacent are generally developed and include residential\commercial\office activities.

4.2 SOIL SURVEY

The Soil Surveys of Miami-Dade County, Florida, as prepared by the U.S. Department of Agriculture, Soil Conservation Service (later renamed the Natural Resource Conservation Service), dated 1967, identifies one soil type at and near the subject site as follows:

15 - Urban Land. This map unit is in areas where more than 85 percent of the surface is covered by shopping centers, parking lots, streets, sidewalks, airports, large buildings, houses, and other structures. The natural soil cannot be observed. The soils in open areas, mostly lawns, vacant lots, playgrounds, and parks, are mainly Udorthents.

These soils generally have been altered by land grading and shaping or have been covered with about 18 inches of extremely stony, loamy fill material. Areas of these soils are so small that mapping them separately is impractical. We note that the maximum depth of the survey is six feet.

The soil survey of Miami-Dade County, Florida from 1947 as prepared by the U.S. Department of Agriculture (USDA) Soil Conservation Service (SCS) was also reviewed. Based on our review, the survey revealed that at the time the survey was conducted, the soils were described as Made Land. This land type was built up from dredging from the bay bottoms in the vicinity of Miami and Miami Beach. Made land is used mainly as building sites for homes, hotels and business establishments.

A USDA Soil Survey Map of the site, Figure 3, is included in Appendix A.

It should be noted that the Soil Survey is not intended as a substitute for site-specific geotechnical exploration; rather it is a useful tool in planning a project scope in that it provides information on soil types likely to be encountered. Boundaries between adjacent soil types on the Soil Survey maps are approximate.

4.3 SUBSURFACE CONDITIONS

4.3.1 REGIONAL GEOLOGY

Miami-Dade County is located in the Coastal Lowlands region of the Florida peninsula. The coastal lowlands consist of nearly level plains, and within Dade County the land surface is generally below Elevation +25 MSL. The surficial soils are comprised of pockets and remnants of Pamlico Sands. The sands are underlain by Miami Limestone (oolitic limestone) followed by limestone and/or sandstone and sand lenses of the Fort Thompson and Tamiami Formations.

The Pamlico Formation is composed of non-fossiliferous, unconsolidated quartz fine sand. Except where outcrops of limestone and man-made fills occur, this formation covers the Miami Limestone. Miami Limestone can be found at or near the surface in the Miami-Dade area. This formation is an oolitic limestone that is generally less than 40 feet thick. It characteristically contains large quantities of ooliths, which are small, spherical particles formed when calcite or aragonite was deposited in concentric layers around a nucleus of some type.

This formation contains solution channels in the limestone which may be up to several feet in diameter at some locations and are filled with quartz fine sand and uncemented calcareous materials. The limestone varies in both thickness and competency within the investigated area. The degree of cementation, and therefore the competency of the rock, was influenced by both the abundance and the type of calcareous material in the original deposit.

The Fort Thompson Formation, which consists of interbedded limestone, sand, and shells, is one of the most productive units within the Biscayne aquifer. It averages 50 to 70 feet in thickness. It typically consists of alternating freshwater and marine sediments, which generally are permeable. The limestone beds in the Fort Thompson Formation can be cavernous and interconnected, thus providing channels through which water can flow.

The Fort Thompson Formation is composed of sediments of variable lithologies. The lithologies include non-fossilferous quartz fine sand, fossilferous quartz sandy limestone, coralline limestone, freshwater limestone and quartz sandstone. These lithologies alternate abruptly in thickness and lateral extent.

4.3.2 STANDARD PENETRATION TEST (SPT)

Our understanding of the subsurface conditions at the project site is derived by performing subsurface explorations, our understanding of geological conditions at the project site, and laboratory testing performed on samples recovered from the project site.

Soil stratification is based on an examination of the recovered soil samples, the laboratory testing, and interpretation of field boring logs by a geotechnical engineer or geologist. The stratification lines represent the approximate boundaries between soil types of significantly different engineering properties. The actual transition may be gradual.

In some cases, small variations in properties not considered pertinent to our engineering evaluation may have been abbreviated or omitted for clarity. The logs represent the conditions at the boring locations only and variations may occur among the borings.

Generalized Site Stratigraphy

STRATUM	SOIL DESCRIPTION	USCS SOIL CLASSIFICATION
0	ASPHALT/TOPSOIL	-
1	Granular FILL comprised of LIMEROCK/SAND/SAND WITH LIMEROCK FRAGMENTS	FILL/SP
2	SILTY SAND	SM
2a	ORGANIC SILTY SAND	OL
3a	LIMESTONE with Fine Sand	
3b	SAND	SP

Standard Penetration Values (N-values) within the upper fill material, Stratum 1, varied from 2 to 44 blows per foot (bpf). N-values in the Silty Sand, Stratum 2, ranged from 4 to 6 bpf and in the organic silty fine sand recorded values of 2 bpf.

N values within the Limestone formation, Stratum 3a, ranged from 8 to 28 blows values in excess of 50 bpf. For Stratum 3b, the sand recorded N-values in the range of 10 to 14 bpf.

For a more detailed description of the subsurface conditions encountered, please refer to the boring logs in Appendix B.

4.4 GROUNDWATER CONDITIONS

Groundwater levels were measured while drilling for the presence and level of groundwater. Groundwater levels observed at these times are indicated on the boring logs. During the subsurface exploration, groundwater was observed in each of the soil borings at depths ranging between 4.5 feet and 7 feet below the existing ground surface.

These groundwater level observations provide an approximate indication of the groundwater conditions existing on the site at the time the borings were drilled. It should be noted that fluctuations in the groundwater table can occur due to seasonal variations, tidal conditions, recent rainfall conditions and other site specific conditions.

5.0 EVALUATIONS AND RECOMMENDATIONS

Based on the results of our study, the subsurface conditions appear to be suitable for the proposed Seawalls to be constructed for North Bay Village.

We note that boring B-2 found the presence of buried organic silty sand soils (OL) at a depth of about 8 feet and extending to a depth of 13 feet below existing ground surface. Organic content measured in the organic silty sand soils was about 7 percent. Based on the laboratory results, these organic soils demonstrate very poor engineering characteristics, most notably low strength and high compressibility and are considered unsuitable. The presence of these unsuitable soils should be taken into consideration in the design of seawalls and seawall installation.

The following sections provide discussions regarding geotechnical recommendations for the construction of the new seawalls.

5.1 Seawall Systems

Conventional seawall systems include sheet piling fabricated from concrete, aluminum, fiberglass, and steel. The type of the sheet pile chosen depends upon a number of factors including both strength and environmental requirements. The designer must consider the possibility of material deterioration and its effect on the structural integrity of the system.

Aluminum and fiberglass offer increased corrosion resistance which is critical in the harsh environment surrounding the seawalls. Most permanent structures are constructed of steel or concrete. Concrete is capable of providing a long service life under normal circumstances but has relatively high initial costs when compared to steel sheet piling. They are more difficult to install than steel piling.

Long-term field observations indicate that steel sheet piling provides a long service life when properly designed. Permanent installations should allow for subsequent installation of cathodic protection before excessive corrosion occurs. These types of seawalls may be cantilevered, tie back, or utilize a king pile/battered pile configuration for additional lateral support.

An evaluation of the field data collected was conducted and geotechnical design parameters were obtained based on the empirical correlations and our experiences. Geotechnical parameters for pile evaluation are provided in the next section of the report.

5.2 Geotechnical Design Parameters

Geotechnical parameters for seawall evaluation shown in the table below were derived empirically using established relationships between the SPT "N" values, soil/rock properties, literature review and our local experience. The following strata encountered during the performance of the field exploration program have been assigned geotechnical parameters. The table below presents a summary of the geotechnical parameters for use in seawall analysis and design.

SUMMARY OF GEOTECHNICAL DESIGN SOIL/ROCK PARAMETERS

Stratum	Material Type	Approximate Layer Thickness (feet) (*)	Unit Weight (pcf)		Friction Angle (Degrees)	Wall Friction Angle, δ (Degrees)	Earth Pressure Coefficients		
			Total	Eff.			Active	Passive	At-Rest
			γ	γ'			Ka	Kp	Ko
1	FILL	8 – 12	115	53	31	23	0.32	3.12	0.48
2a	SILTY SAND	5.5 – 9	105	43	26	20	0.39	2.56	0.56
2b	ORGANIC SILTY SAND	4	80	18	-	-	-	-	-
3	LIIMESTONE	12.5 – 16.5	120	58	37	28	0.25	4.02	0.40
3a	SAND	5	108	46	30	23	0.33	3.00	0.50

Notes:
Depths measured from existing grade at time of boring.
Groundwater shall be assumed at the ground surface for calculation design purposes
Refer to the boring logs for Complete Soil Description
At rest earth pressure, K_O, is calculated as 1 - sin Ø for sands.
Friction angle between concrete and soil should be taken as d = 3/4 Ø (NAVFAC DM-7.2)
Limestone layers modeled as sand to sandy gravel for estimation of friction angle, Ø. Friction Angle, φ = N/4 + 33.
Strata 0 are the asphalt pavement and topsoil. This top layer should be ignored for calculation purposes.

5.3 DESIGN OF PROPOSED SEAWALL

We understand that a seawall will be constructed at the shorelines near the intersections indicated in section 1.2. The proposed seawall may be designed using the soil/rock parameters presented in Section 5.2 of this report. The type and design of seawalls was not included in our scope of services and we are assuming this will be performed by others.

6.0 CONSTRUCTION CONSIDERATIONS

The following are our suggestions for the installation of the proposed seawall based on the results of the test borings.

It should be noted that sheet pile refusal may occur on a random and unpredictable basis since zones of dense rock/soils may be encountered. In this case, we recommend that predrilling be considered prior to the installation of the sheet piles. Predrilling is required in order to prevent refusal conditions, damage of the structural section of the sheeting and minimize vibrations-induced settlements to nearby structures. Following predrilling. the sheet piles should be set in place and vibrated to the required tip elevations.

The sheet pile installation equipment will produce vibration and noise levels that may be considered disturbing to people and can produce vibrations noticeable in structures. The potential for damage to any adjacent structures during the sheet pile installations will be dependent on the distance from the adjacent structures to the location of the sheet piles installation, the subsurface conditions, and the level of sensitivity of the structure to any type of vibration. The recommendations provided in Section 455-1.1 in the latest version of the FDOT Standard Specifications for Road and Bridge Construction should be followed for the protection of the existing structures during sheet piling operations. All those structures and or utilities located adjacent to the proposed excavation shall be surveyed as well as monitored for vibrations and settlements in accordance with Section 455-1.1 of the latest version of the FDOT Standard Specifications for Road and Bridge Construction

The select fill used as backfill should be tested and approved prior to acquisition and placement. Fill materials required at the sites should consist of clean sand, a mixture of sand and limerock fragments (SP, GP), free of organic matter and debris. The fill should be non-plastic, with a fines content of less than 5 percent. The water content of the soil at the time of compaction should be within +/-2 percent of the soil's optimum moisture content as determined by the Modified Proctor Test (ASTM D-1557 or AASHTO T-180). Fill materials should be placed and compacted in lifts not exceeding 12-inch loose layers.

Prior to initiating compaction operations, we recommend that representative samples of the select fill material to be used and acceptable in-place soils be collected and tested to determine their compaction and classification characteristics. The maximum dry density, optimum moisture content, gradation, and plasticity characteristics should be determined. These tests are needed for compaction quality control of the select fill and existing soils and to determine if the fill material is acceptable. Density tests to confirm compaction should be preformed in each fill lift before the next lift is placed. Any fill indicating less than above compaction requirements should be recompacted until the required density is obtained.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean that GCES is assuming any responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

7.0 ADDITIONAL CONSTRUCTION CONSIDERATIONS

7.1 QUALITY ASSURANCE

We recommend establishing a comprehensive quality assurance program to verify that all foundation construction is conducted in accordance with the appropriate plans and specifications. Materials testing and inspection services should be provided by GCES. In-situ density tests should be conducted during backfilling activities to verify that the required densities have been achieved. In-situ density values should be compared to laboratory Proctor moisture-density results for each of the different natural and fill soils encountered.

7.2 EXCAVATIONS

All excavations should be sloped or braced as required by OSHA regulations to provide stability and safe working conditions. We recommend trench excavations in excess of 5 feet be supported with temporary shoring and bracing, such as trench boxes. The grading contractor, by his contract, is usually responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. All excavations should comply with applicable local, state and federal safety regulations, including the current Occupational Health and Safety Administration (OSHA) Excavation and Trench Safety Standards. GCES does not assume any responsibility for excavation or construction site safety, or the contractors activities; such responsibility shall neither be implied nor inferred.

8.0 LIMITATIONS

The evaluations presented in this Report of Geotechnical Exploration were prepared for exclusive use of EAC Consulting, Inc. for specific application of the North Bay Village Seawalls in North Bay Village, Miami-Dade County, FL. The scope of investigation was intended to specifically evaluate subsurface conditions within the influence of the proposed structures mentioned herein. These evaluations and recommendations were prepared using generally accepted standards of geotechnical engineering practices. No other warranty is expressed or implied.

Our geotechnical engineering evaluation of the site and subsurface conditions with respect to structures submittal and our recommendations are based upon the following: 1) site observations; 2) the field exploratory test data obtained during this phase of the study, and 3) our understanding of the project information as presented in this report.

Since this is an exploration, further consultation with GCES during the design process will be required so that these recommendations can be adjusted to the actual design. Furthermore, upon the discovery of any site or subsurface condition during construction which appears to deviate from the data presented and documented herein, please contact us immediately so that we may visit the site, observe the differing conditions, and thus evaluate this new information concerning these recommendations.

The recommendations presented represent design information that GCES believes are both applicable and feasible for the planned construction and as noted above, it is based on the information provided to GCES as summarized.

Involvement of the geotechnical engineer during the design process and subsequently with the construction process is vitally important to ensure the project is constructed in accordance with the recommendations from the geotechnical report. Should subsurface changes be encountered, early involvement of the geotechnical engineer can hasten subsequent recommendations. In addition, if varying subsurface conditions are encountered, resolutions can be obtained more quickly.

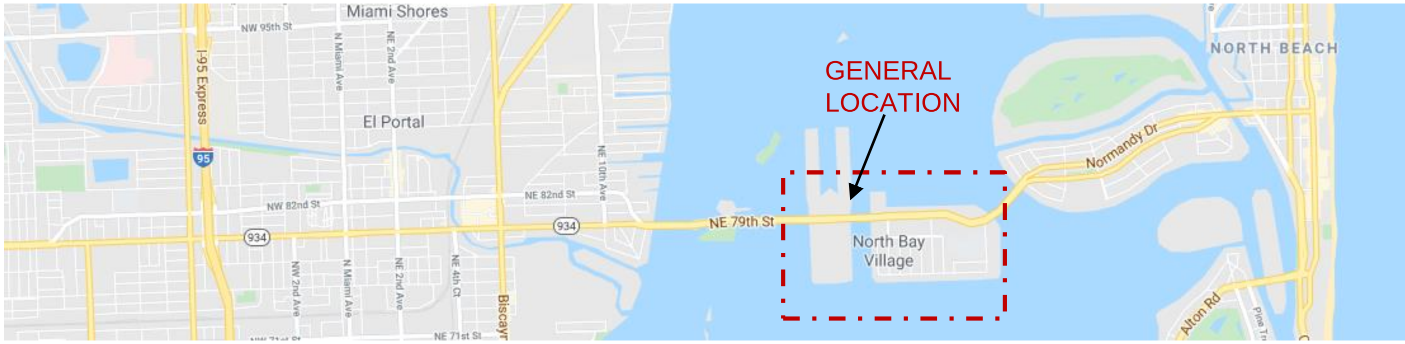
The assessment of site environmental conditions for the presence of contaminants in the soil, rock, surface, or groundwater of the site was beyond the scope of this exploration.

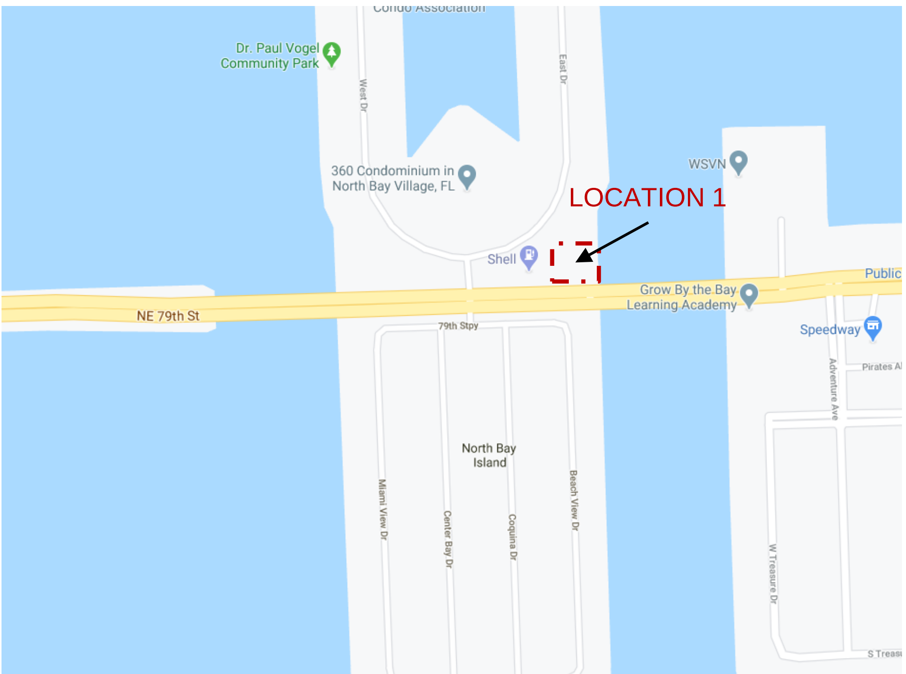
APPENDIX A

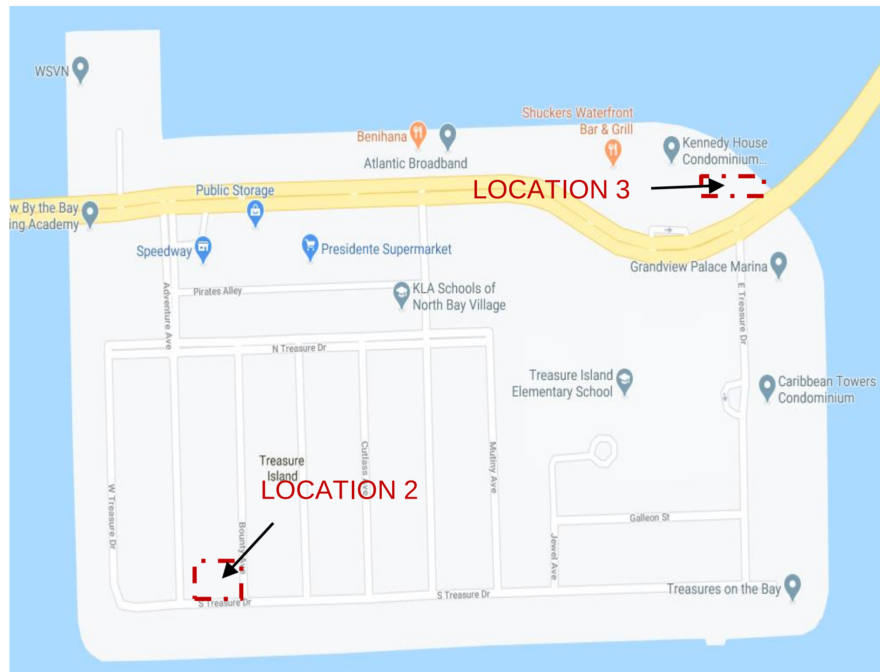
VICINITY MAP – FIGURE 1

BORING LOCATION PLAN – FIGURE 2

SOIL SURVEY MAP – FIGURE 3







LEGEND

VICINITY MAP

Reference: http://www.googlemap.com

Subject Sites

Project Manager: ARM	Project No. G10201005
Drawn by: DC	Scale: N.T.S.
Checked by: ARM	File Name:
Approved by: ARM	Date: 03/17/2020

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VICINITY MAP

Geotechnical Engineering Services
North Bay Village Seawalls
North Bay Village, Miami Dade County, FL

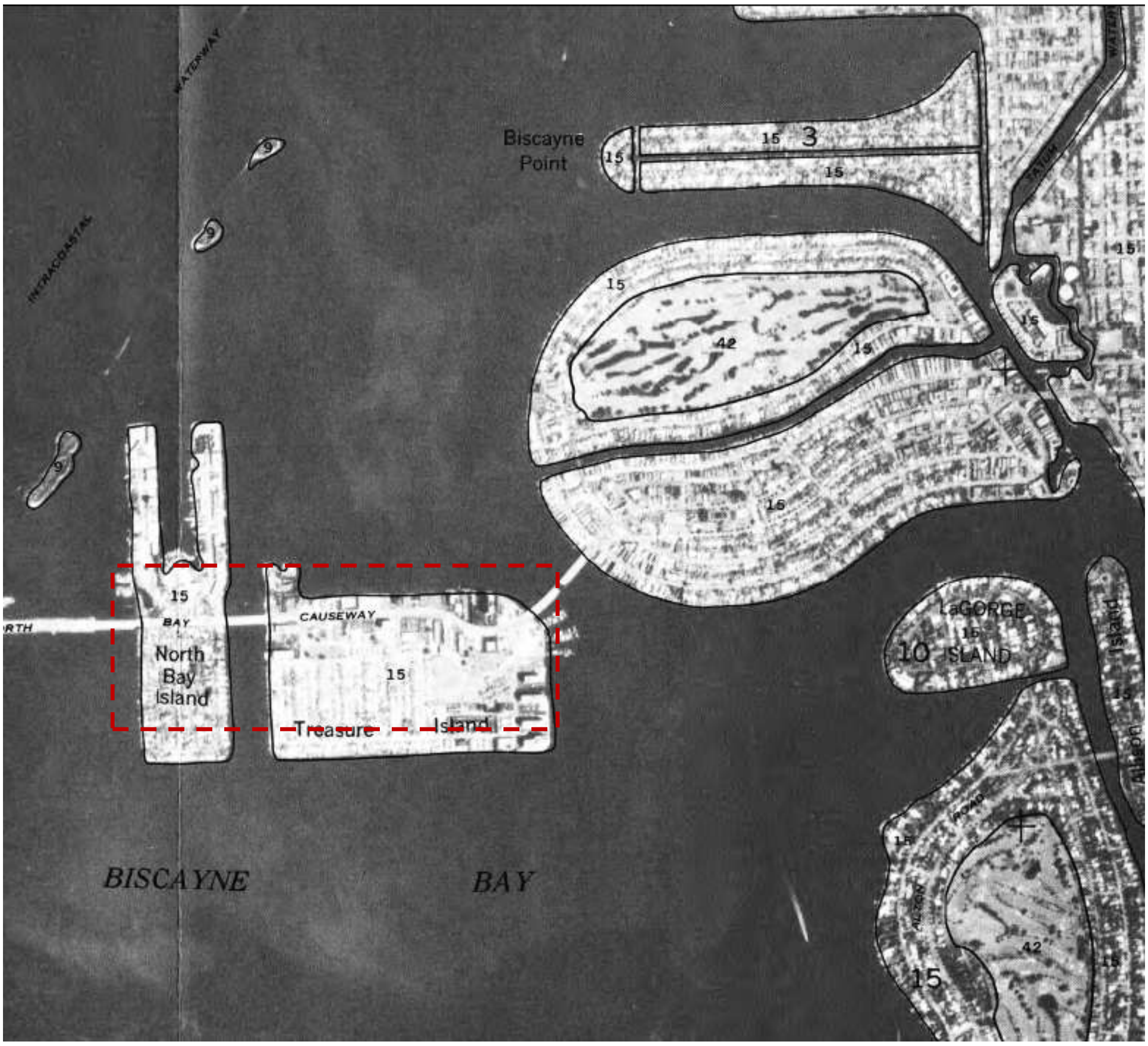
FIG 1

LEGEND

Approximate Boring Location

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: ARM	Project No. G10201005	GCES GEOTECHNICAL CONSTRUCTION & ENGINEERING SOLUTIONS	BORING LOCATION PLAN
Drawn by: DC	Scale: N.T.S.		Geotechnical Engineering Services North Bay Village Seawalls North Bay Village, Miami Dade County, FL
Checked by: ARM	File Name:		
Approved by: ARM	Date: 03/17/2020		FIG 2



SOIL SURVEY MAP

From U.S. Department of Agriculture, Soil Conservation Service (later renamed the Natural Resource Conservation Service), dated 1967

LEGEND

SUBJECT SITE

15 - Urban Land Complex

Site Boundaries Are Approximate

Project Manager: ARM	Project No. G10201005	GCES GEOTECHNICAL CONSTRUCTION & ENGINEERING SOLUTIONS	FIG. No. 3 Geotechnical Engineering Services North Bay Village Seawalls North Bay Village Miami Dade County, FL
Drawn By: DC	Scale: N.T.S.		
Checked By: ARM	Date:03/17/2020		

APPENDIX B

Borings Logs – (B-1 through B-3)

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BORING NUMBER B- 1

PROJECT NUMBER G10201005

CLIENT EAC CONSULTING, INC

PROJECT NAME North Bay Village Seawalls

PROJECT LOCATION NEQ of Intersection of East Drive and John F. Kennedy Cswy, North Bay Village, Florida

DATE STARTED 3/10/2020 **COMPLETED** 3/10/2020

SURFACE ELEVATION REFERENCE Above road crown

DRILLING METHOD Standard Penetration Boring

LOGGED BY L.T. **CHECKED BY** ARM

GROUND WATER LEVELS AT TIME OF DRILLING 6 feet

APPROXIMATE LOCATION OF BORING See site plan

DEPTH	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOWS	N-VALUE
2	6 Inches - TOPSOIL - Tan Fine SAND Trace Limerock Fragments and Roots, Medium Dense (SP, FILL)	SS 1	4 6 6 7	12
4	Light Brown Fine SAND with Limerock Fragments, Medium Dense (SP, FILL)	SS 2	6 7 7 9	14
5	Light Brown LIMEROCKFragments with Fine Sand, Loose to Medium Dense (SP, FILL)	SS 3	7 7 6 6	13
		SS 4	6 4 4 4	8
10		SS 5	6 4 4 4	8
12				
15	Gray SILTY Fine SAND with Shell, Loose (SM) N.M.C= 33% O.C.=3% -200=24%	SS 6	3 3 3 3	6
17		SS 7	3 2 2 1	4
20	Gray to Tan LIMESTONE with Fine Sand and Silt	SS 8	4 4 4 6	8

SPT N VALUE: 10 20 30 40 50
PL MC LL: 20 40 60 80
FINES CONTENT %

Continue Next Page

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BORING NUMBER B- 1

PROJECT NUMBER G10201005

CLIENT EAC CONSULTING, INC

PROJECT NAME North Bay Village Seawalls

PROJECT LOCATION NEQ of Intersection of East Drive and John F. Kennedy Cswy, North Bay Village, Florida

DATE STARTED 3/10/2020 **COMPLETED** 3/10/2020

SURFACE ELEVATION REFERENCE Above road crown

DRILLING METHOD Standard Penetration Boring

LOGGED BY L.T. **CHECKED BY** ARM

GROUND WATER LEVELS AT TIME OF DRILLING 6 feet

APPROXIMATE LOCATION OF BORING See site plan

DEPTH	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOWS	N-VALUE	SPT N VALUE (10 20 30 40 50) / PL MC LL (20 40 60 80) / FINES CONTENT %
		Tan LIMESTONE, Trace Fine Sand and Silt				
25			SS 9	6 7 5 11	12	
30			SS 10	4 6 6 8	12	
		Gray Fine SAND, Medium Dense, (SP)				
35			SS 11	6 6 4 6	10	
		Boring Terminated @ 35 feet				

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BORING NUMBER B- 2

PROJECT NUMBER G10201005

CLIENT EAC CONSULTING, INC **PROJECT NAME** North Bay Village Seawalls

PROJECT LOCATION NWQ of Intersection of Bounty Avenue and South Treasure Drive, North Bay Village, Florida

DATE STARTED 3/10/2020 **COMPLETED** 3/10/2020 **SURFACE ELEVATION REFERENCE** Same road crown

DRILLING METHOD Standard Penetration Boring

LOGGED BY L.T. **CHECKED BY** ARM **GROUND WATER LEVELS AT TIME OF DRILLING** 4.6 feet

APPROXIMATE LOCATION OF BORING See site plan

DEPTH		GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOWS	N-VALUE	SPT N VALUE (10 20 30 40 50); PL MC LL (20 40 60 80); FINES CONTENT %
	1		1 Inches - ASPHALT - Tan LIMEROCK Fragments with Fine Sand, Dense (SP, FILL)	SS 1	27 16 16 14	32	▲
	2		Brown LIMEROCK Fragments with Fine Sand and Shell, Dense (SP, FILL)				
			Gray SILTY Fine SAND, Very Loose to Loose (SM) N.M.C= 41% O.C.=3% -200=39%	SS 2	7 4 3 2	7	▲
5	▼			SS 3	2 1 1 1	2	▲
	8			SS 4	1 1 1 1	2	▲
10			Gray ORGANIC SILTY Fine SAND, Very Loose (OL) N.M.C=57% O.C.=7% -200=65%	SS 5	1 1 1 1	2	▲
	13			SS 6	1 1 1 1	2	▲
15			Gray SILTY Fine SAND with Limestone Fragments, Trace Shell, Medium Dense (SM)	SS 7	4 6 6 6	12	▲
	17		Tan LIMESTONE, Trace Fine Sand and Silt				
20				SS 8	10 14 10 11	24	▲

Continue Next Page

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BORING NUMBER B- 2

PROJECT NUMBER G10201005

CLIENT EAC CONSULTING, INC
PROJECT NAME North Bay Village Seawalls
PROJECT LOCATION NWQ of Intersection of Bounty Avenue and South Treasure Drive, North Bay Village, Florida
DATE STARTED 3/10/2020 **COMPLETED** 3/10/2020
DRILLING METHOD Standard Penetration Boring
SURFACE ELEVATION REFERENCE Same road crown
LOGGED BY L.T. **CHECKED BY** ARM
GROUND WATER LEVELS AT TIME OF DRILLING 4.6 feet
APPROXIMATE LOCATION OF BORING See site plan

DEPTH		GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOWS	N-VALUE	▲ SPT N VALUE ▲ (10 20 30 40 50) / PL MC LL (20 40 60 80) / FINES CONTENT %
			Tan LIMESTONE, Trace Fine Sand and Silt				
25				SS 9	8 10 10 10	20	▲
	27		Light Gray Fine SAND, Trace Limestone Fragments, Medium Dense (SP)				
30				SS 10	4 6 6 7	12	▲
	31		Light Gray Fine SAND, Medium Dense, (SP)				
35				SS 11	6 7 7 9	14	▲
			Boring Terminated @ 35 feet				

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GEOTECHNICAL CONSTRUCTION & ENGINEERING SOLUTIONS

BORING NUMBER B- 3

PROJECT NUMBER G10201005

CLIENT EAC CONSULTING, INC
PROJECT NAME North Bay Village Seawalls
PROJECT LOCATION NWQ of Intersection of John F. Kennedy Cswy and E. Treasure Drive, North Bay Village, Florida
DATE STARTED 3/10/2020 **COMPLETED** 3/10/2020
SURFACE ELEVATION REFERENCE Below road crown
DRILLING METHOD Standard Penetration Boring
LOGGED BY L.T. **CHECKED BY** ARM
GROUND WATER LEVELS AT TIME OF DRILLING 7 feet
APPROXIMATE LOCATION OF BORING See site plan

DEPTH	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOWS	N-VALUE
0–1	1 Inches - ASPHALT - Light Brow Fine SAND with Limerock Fragments, Medium Dense (SP, FILL)	SS 1	26 24 20 17	44
1–2	Light Brown Fine SAND, Trace Limerock Fragments, Medium Dense (SP, FILL)			
2–8	Light Brown Fine SAND, Medium Dense (SP, FILL)	SS 2	7 7 7 6	14
5		SS 3	6 7 7 6	14
		SS 4	6 7 7 4	14
8–17	Gray SILTY Fine SAND, Trace Shell, Loose (SM) N.M.C.=38% O.C.=3% --200=20%	SS 5	4 2 2 2	4
10				
15		SS 6	2 2 2 3	4
17–20	Gray SILTY Fine SAND, Trace Limestone Fragments, Medium Dense (SM)	SS 7	8 13 7 11	20
20				

SPT N VALUE: 10 20 30 40 50
PL MC LL: 20 40 60 80
FINES CONTENT %

Continue Next Page

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BORING NUMBER B- 3

PROJECT NUMBER G10201005

CLIENT EAC CONSULTING, INC
PROJECT NAME North Bay Village Seawalls
PROJECT LOCATION NWQ of Intersection of John F. Kennedy Cswy and E. Treasure Drive, North Bay Village, Florida
DATE STARTED 3/10/2020 **COMPLETED** 3/10/2020
SURFACE ELEVATION REFERENCE Same road crown
DRILLING METHOD Standard Penetration Boring
LOGGED BY L.T. **CHECKED BY** ARM
GROUND WATER LEVELS AT TIME OF DRILLING 7 feet
APPROXIMATE LOCATION OF BORING See site plan

DEPTH		GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOWS	N-VALUE	▲ SPT N VALUE ▲ (10 20 30 40 50); PL MC LL (20 40 60 80); FINES CONTENT %
			Tan LIMESTONE, Trace Fine Sand and Silt				
25				SS 9	11 14 14 10	28	▲
30				SS 10	8 10 10 10	20	▲
	31		Light Brown Fine SAND with Limestone, Medium Dense, (SP)				
35				SS 11	6 6 8 10	14	▲
			Boring Terminated @ 35 feet				

APPENDIX C

GENERAL NOTES

UNIFIED SOIL CLASSIFICATION SYSTEM

FIELD EXPLORATORY DESCRIPTION

LABORATORY TESTING DESCRIPTION

GENERAL NOTES

DRILLING & SAMPLING SYMBOLS:

SS:	Split Spoon - 1-^{3}/8" I.D., 2" O.D., unless otherwise noted	HS:	Hollow Stem Auger
ST:	Thin-Walled Tube - 2" O.D., unless otherwise noted	PA:	Power Auger
RS:	Ring Sampler - 2.42" I.D., 3" O.D., unless otherwise noted	HA:	Hand Auger
DB:	Diamond Bit Coring - 4", N, B	RB:	Rock Bit
BS:	Bulk Sample or Auger Sample	WB:	Wash Boring or Mud Rotary

The number of blows required to advance a standard 2-inch O.D. split-spoon sampler (SS) the last 12 inches of the total 18-inch penetration with a 140-pound hammer falling 30 inches is considered the “Standard Penetration” or “N-value”. For 3” O.D. ring samplers (RS) the penetration value is reported as the number of blows required to advance the sampler 12 inches using a 140-pound hammer falling 30 inches, reported as “blows per foot,” and is not considered equivalent to the “Standard Penetration”or “N-value”.

WATER LEVEL MEASUREMENT SYMBOLS:

WL:	Water Level	WS:	While Sampling	N/E:	Not Encountered
WCI:	Wet Cave in	WD:	While Drilling		
DCI:	Dry Cave in	BCR:	Before Casing Removal		
AB:	After Boring	ACR:	After Casing Removal		

Water levels indicated on the boring logs are the levels measured in the borings at the times indicated. Groundwater levels at other times and other locations across the site could vary. In pervious soils, the indicated levels may reflect the location of groundwater. In low permeability soils, the accurate determination of groundwater levels may not be possible with only short-term observations.

DESCRIPTIVE SOIL CLASSIFICATION: Soil classification is based on the Unified Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

CONSISTENCY OF FINE-GRAINED SOILS

Unconfined Compressive Strength, Qu, psf	Standard Penetration or N-value (SS) Blows/Ft.	Consistency
< 500	<2	Very Soft
500 – 1,000	2-3	Soft
1,001 – 2,000	4-6	Medium Stiff
2,001 – 4,000	7-12	Stiff
4,001 – 8,000	13-26	Very Stiff
8,000+	26+	Hard

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Standard Penetration or N-value (SS) Blows/Ft.	Ring Sampler (RS) Blows/Ft.	Relative Density
0 – 3	0-6	Very Loose
4 – 9	7-18	Loose
10 – 29	19-58	Medium Dense
30 – 49	59-98	Dense
50+	99+	Very Dense

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 – 29
Modifier	> 30

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 – 12
Modifiers	> 12

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300mm)
Cobbles	12 in. to 3 in. (300mm to 75 mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm)
Silt or Clay	Passing #200 Sieve (0.075mm)

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1-10
Medium	11-30
High	30+

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]					Soil Classification: Group Symbol	Soil Classification: Group Name[B]
Coarse Grained Soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[E]		GW	Well-graded gravel[F]
			$Cu < 4$ and/or $1 > Cc > 3$[E]		GP	Poorly graded gravel[F]
		Gravels with Fines More than 12% fines[C]	Fines classify as ML or MH		GM	Silty gravel[F,G,H]
			Fines classify as CL or CH		GC	Clayey gravel[F,G,H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$[E]		SW	Well-graded sand[I]
			$Cu < 6$ and/or $1 > Cc > 3$[E]		SP	Poorly graded sand[I]
		Sands with Fines More than 12% fines[D]	Fines classify as ML or MH		SM	Silty sand[G,H,I]
			Fines Classify as CL or CH		SC	Clayey sand[G,H,I]
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	inorganic	$PI > 7$ and plots on or above "A" line[J]		CL	Lean clay[K,L,M]
			$PI < 4$ or plots below "A" line[J]		ML	Silt[K,L,M]
		organic	Liquid limit - oven dried	< 0.75	OL	Organic clay[K,L,M,N]
			Liquid limit - not dried			Organic silt[K,L,M,O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line		CH	Fat clay[K,L,M]
			PI plots below "A" line		MH	Elastic Silt[K,L,M]
		organic	Liquid limit - oven dried	< 0.75	OH	Organic clay[K,L,M,P]
			Liquid limit - not dried			Organic silt[K,L,M,Q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor				PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains $\geq$ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains $\geq$ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains $\geq$ 30% plus No. 200 predominantly sand, add "sandy" to group name.

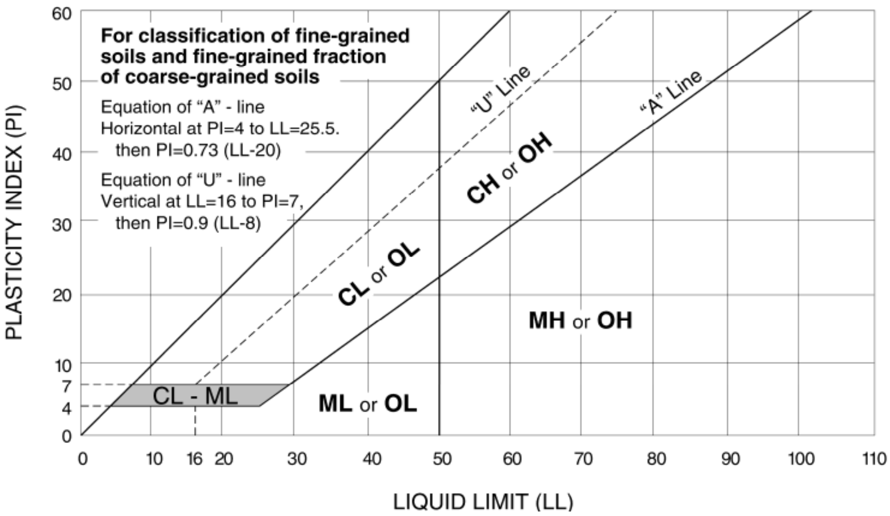
[M] If soil contains $\geq$ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] $PI \geq 4$ and plots on or above "A" line.

[O] $PI < 4$ or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



FIELD EXPLORATORY DESCRIPTION

Standard Penetration Test (SPT)

Soil samples were obtained by the split spoon sampling procedure in general accordance with the Standard Penetration Test (SPT) procedure ASTM Standard D-1586. The SPT procedure consists of driving a split-barrel sampler to obtain a soil sample and to measure the resistance (N-value) of the soil to penetration of the sampler. In the split barrel sampling procedure, the number of blows required to advance a standard 2 inch O.D. split barrel sampler the last 12 inches of an 18-inch penetration or the middle 12 inches of a 24-inch penetration by means of a 140 pound hammer with a free fall of 30 inches, is the standard penetration resistance value (N).

The N-values provide a measure of the relative density of cohesionless soils (sands) and the consistency of cohesive soils (clays) sampled during drilling. Engineering properties of the soils are inferred from SPT N-values and index property soil classification, based on published empirical correlations.

The N-values also provide a general indication of hardness for rock formations such as the limestone commonly encountered in the Southeast Florida area. Where limestone is encountered, the Standard Penetration Test is used as a general indication of hardness. Where low blows per foot are encountered, it is assumed that solution cavities filled with loose sands or soft silt soils are present within the limestone formation.

LABORATORY TESTING PROCEDURE

Percent Passing No. 200 Sieve

The grain size analysis were conducted in general accordance with FDOT test Designation (FM-1-T88 (ASTM Designation D-422, tilted "Particle Side Analysis of Soils"). The grain-size analysis test measures the percentage passing the No. 200 Sieve. In this manner, the grain-size distribution of a soil is measured. The percentage by weight passing the No. 200 Sieve is the amount of silt and clay sized particles. Other samples were analyzed for fines content only by measuring the percentage by weight of dry soil sample passing a U.S. standard No. 200 sieve in general accordance with ASTM-D1140.

Moisture Content

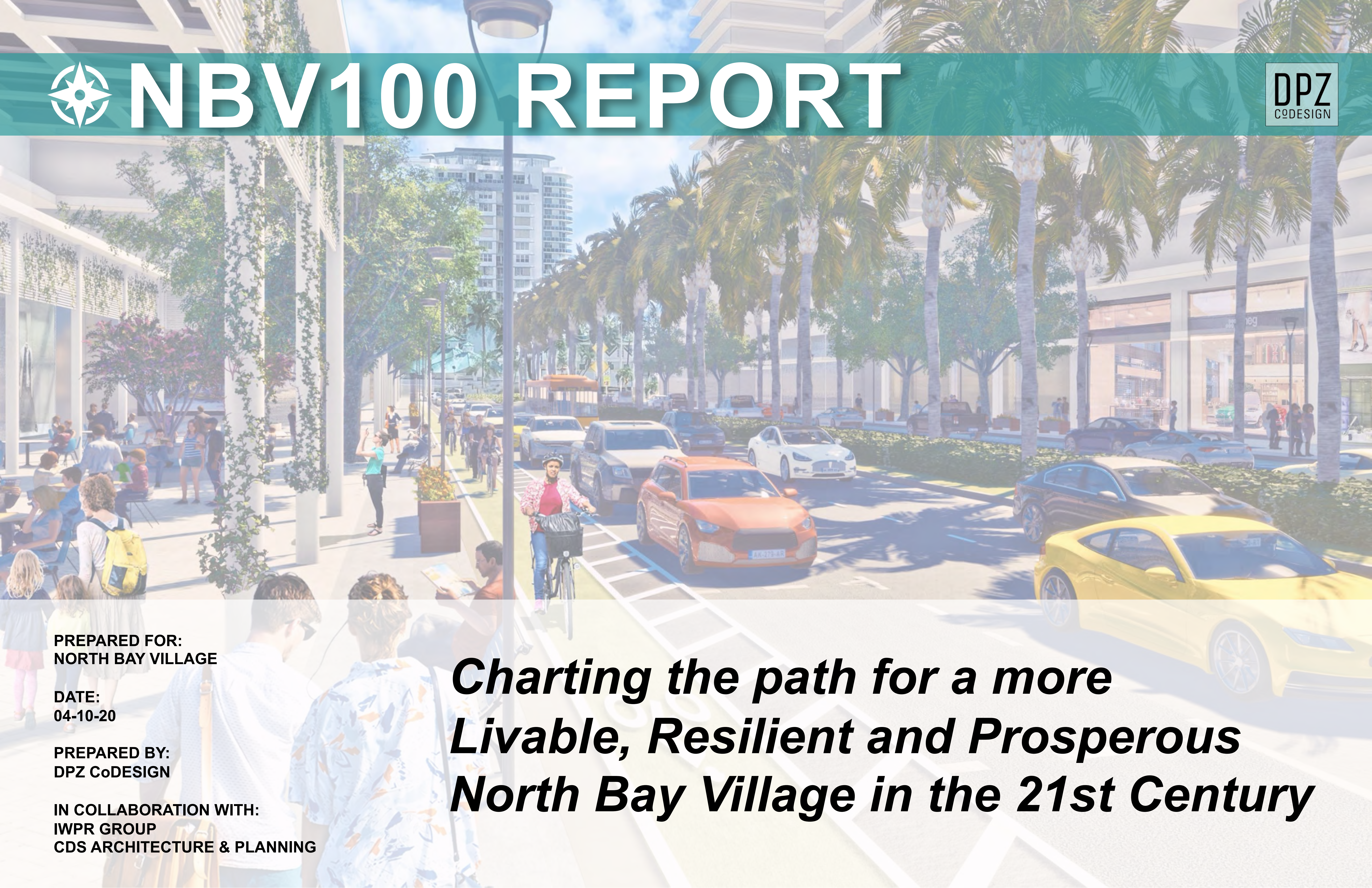
In order to determine the moisture content of soil samples, test specimens were dried in an oven to constant mass in general accordance with ASTM-D2216. The water content is then calculated using the mass of the water and the mass of the dry specimen. The water content is used to express the phase relationship of air, water, and solid in a given volume of material. In fine grained soils, the consistency of a given soil type depends on its water content.

Organic Content

In order to determine the compressibility of soil over time, organic content tests were performed on soil sample collected from soil layers suspected of containing significant amounts of organic materials. Organic content is determined by methods similar to those employed to find water content. The dry test specimen is burnt in a hot oven until it reaches a constant mass. The loss of mass due to burning is considered to be organic materials in the soil. The organic soil content is then calculated using the mass of the organics and the mass of the burnt specimen.

Appendix C

NBV100: Embracing the Waterfront



NBV100 REPORT

DPZ CoDESIGN

PREPARED FOR:
NORTH BAY VILLAGE

DATE:
04-10-20

PREPARED BY:
DPZ CoDESIGN

IN COLLABORATION WITH:
IWPR GROUP
CDS ARCHITECTURE & PLANNING

Charting the path for a more Livable, Resilient and Prosperous North Bay Village in the 21st Century

EMBRACE THE WATERFRONT

Enable public accessibility and walkability of
NBV's privileged island waterfront

Treasure Island Waterfront

Harbor Island Waterfront

EMBRACE THE WATERFRONT

PRIOR AND EXISTING INITIATIVES FOR PUBLIC SHORELINE ACCESS: THE BAY WALK

The Village has been seeking to improve connections with Biscayne Bay for a number of years. For some time, existing NBV zoning regulations have required that new developments provide a shoreline walkway. Within the 25-foot setback mandated by the County on all waterfront properties, a 10-foot wide easement for public shoreline access has been required. This applies whenever waterfront properties get redeveloped with multi-family or mixed-use projects. In 2018, this requirement was extended to purely commercial waterfront projects as well, which includes hotels. As properties get redeveloped over time, this will eventually result in a continuous path around most of Treasure Island and all of Harbor Island – indeed, everywhere in NBV, except in the single-family districts. Previously, this initiative was sometimes called the Bay Walk.

A number of properties have complied over the years and the Village maintains a list of all easements on record. Unfortunately, many of them are still discontinuous, and, according to some residents who raised their concerns during the charrette, not all are open to the public, as they should be. Complaints were made about enforcement.

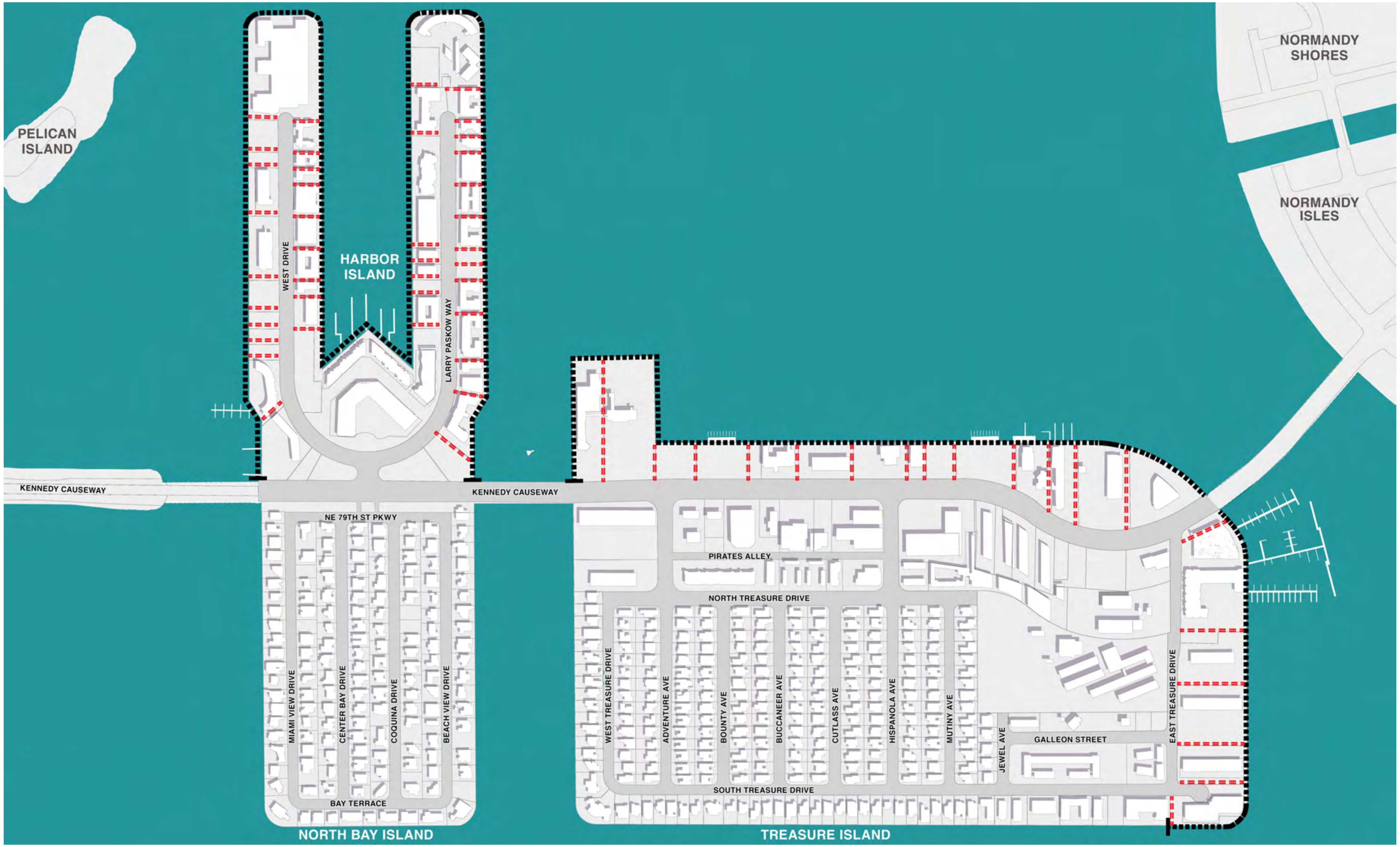
In addition to these zoning requirements, the Village has previously attempted to find funding to expedite the construction of a continuous stretch of the shoreline walkway north of Kennedy. This ambitious scheme would have run continuously both on land and over water with numerous boat docks. While this effort appears to have received some preliminary blessings from local regulators, who generally recognize that NBV is in need of greater public access to the water, adequate funding was never located. In any case, it is not clear that locating so much of the project within the riparian right-of-way would ever have been approved by the County.

It turns out that the existing requirement for a 10-foot wide easement is a bit narrow. Take, for example, the Biscayne Bay Path in Miami Beach. (See photo this page.) Though a nice amenity, it is close to 10 feet in width and feels at bit tight. It lacks space for plantings and benches, let alone outdoor restaurant seating. (See photo.) It can afford to be narrow because Miami Beach already offers so much public access to the water in the form of the beaches, Lummus Park, South Pointe Park and the Boardwalk. By contrast, it is widely acknowledged that NBV is starved for water access. The Island Walk needs to shine.

The recently opened Benihana implemented the existing shoreline access requirements, which is 10 feet wide.

Biscayne Bay Path in Miami Beach features a narrow easement without adequate space for plantings and benches.

NBV SHORELINE WALKWAY



NBV has been seeking to create more connections with Biscayne Bay for several years. Under the existing land use regulations, as properties get redeveloped, a public easement along the water is granted to the Village for a public shoreline walkway. In addition, each property must provide a 5-foot wide public access connection walkway from the street. A number of easements are already on record with the Village. Eventually, much of Harbor Island and Treasure Island will be ringed. This incremental approach is similar to that of the Riverwalk and to the waterfront access required by Maimi21, the zoning regulations of the City of Miami.

As part of NBV100, shoreline access is being reenvisioned as the Island Walk with improved standards and amenities. The Village is seeking grants to expedite its implementation. See Livability for a full description.

Shoreline Walkway – Segment lengths when completed (approximate)

- Treasure Island north of Kennedy 0.81 miles (4,296 FT)
- Treasure Island south of Kennedy 0.45 miles (2,400 FT)
- Harbor Island 1.81 miles (9,567 FT)

Shoreline Walkway (10’)

Access Walkway on Each Property (5’)

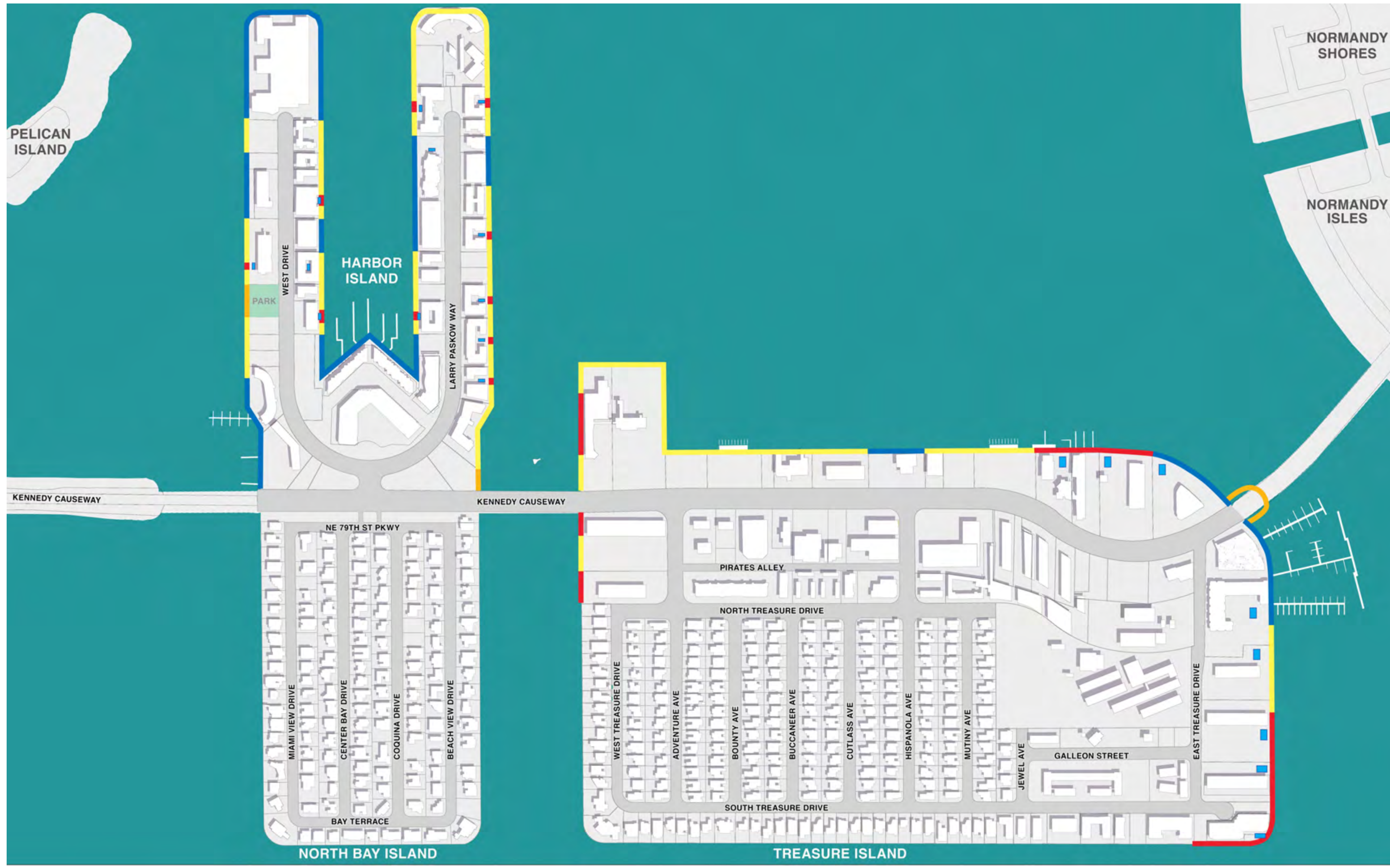
EMBRACE THE WATERFRONT

PRIOR AND EXISTING INITIATIVES FOR PUBLIC SHORELINE ACCESS: THE BAY WALK

Several developers who have been interested in investing in NBV for years have expressed positive sentiments about the idea of a public shoreline walkway. In fact, they view it as a positive amenity that would increase the value of their properties. Indeed, they produced renderings in recent years for some hypothetical projects along Kennedy that feature an on-land version of the walkway prominently.

NBV secured funding for the design of a waterfront pedestrian bridge under the Causeway that would make a difficult connection between two portions of a future walkway.

Preliminary Design for Waterfront Pedestrian Bridge under the Causeway to connect future sections of the Island Walk.
Design by Kimley Horn.



Note: This diagram has been prepared based on observations of Google Earth and Google Maps imagery. As such, some properties may be mischaracterized. Actual conditions should be verified.

- Private Land - Completely Built
- Public Land - Completely Built
- Private Land - Unobstructed Implementation
- Public Land - Unobstructed Implementation
- Private Land - Difficult to Implement
- Public Land - Difficult to Implement
- Pool

EMBRACE THE WATERFRONT

ISLAND WALK: INTRODUCTION

The Village has decided to take a well-intentioned initiative for shoreline access and make it better, weaving it into the NBV100 vision. Even before the charrette began, the Village rebranded the shoreline walkway as the Island Walk. The previous name was too similar to amenities offered by neighboring communities. The new name calls attention to the distinct nature of North Bay Village as a collection of three small islands.

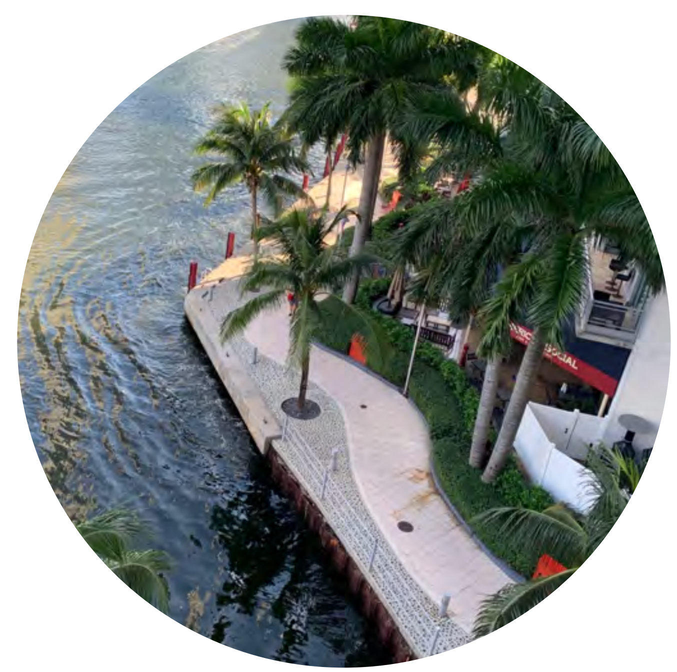
The proposed design for the new Island Walk builds on the Miami21 Waterfront Design Guidelines written in 2009 and the current North Bay Village Waterfront Standards. The width of the Island Walk easement will be increased from 10 feet to 18 feet overall so that it will afford a more pleasant pedestrian experience as well as a variety of other experiences that are desired by the residents of NBV.

Restaurants and retail will be encouraged to front the new Island Walk. Outdoor seating for restaurants will enliven the experience. To facilitate this and other creative uses of the space, flexibility on the placement of the 18 ft. easement within the 25 ft. setback will be allowed. The default location of the easement will typically be adjacent to the water, but it can meander anywhere within the setback. This will allow for other supporting uses, such as small kiosks for food and retail. In some cases, there may be restaurant seating along the water's edge.

In addition, property owners will be encouraged to secure underwater land leases from the State of Florida to construct docks which can be used for restaurant seating, while kitchens and other facilities will remain in the buildings behind the setback, similar to Shuckers, an NBV institution and one of the oldest and most beloved waterfront restaurants in the area.

It is always preferable to construct the Island Walk on land, but this may not be possible in some cases because of existing obstructions. These might include buildings or swimming pools built within the setback. In these cases, an over-water version may be proposed.

Precedent –
South Pointe Bay Walk, FL



Precedent –
Riverwalk in Miami

Precedent –
Riverwalk in Miami

EMBRACE THE WATERFRONT

ISLAND WALK: PRECEDENT IMAGES – SEATING SEPARATED BY A PEDESTRIAN PATH

Both locally and around the world, some of the best places to eat are restaurants with outdoor seating that happens to be separated from the kitchen by a pedestrian path. Waitstaff routinely cross the path without incident. The eateries along Lincoln Road and Smith & Wollensky in South Pointe Park are excellent examples. Note that, in addition to pedestrians, bicyclists and skateboarders routinely use the path in South Pointe, and vehicles use the street in the Sicilian example below. They instinctively slow down in these zones. Private docks with boat slips can also be accessed easily from the Island Walk. Access can be controlled with simple chains or gates.

*Lincoln Road,
Miami Beach*

*South Pointe Park,
Miami Beach*

Sicily, Italy

Venice, Italy

*Giralda Avenue,
Coral Gables*

EMBRACE THE WATERFRONT

THE ISLAND WALK: DETAILING AND DIMENSIONING OF WATERFRONT PROMENADE

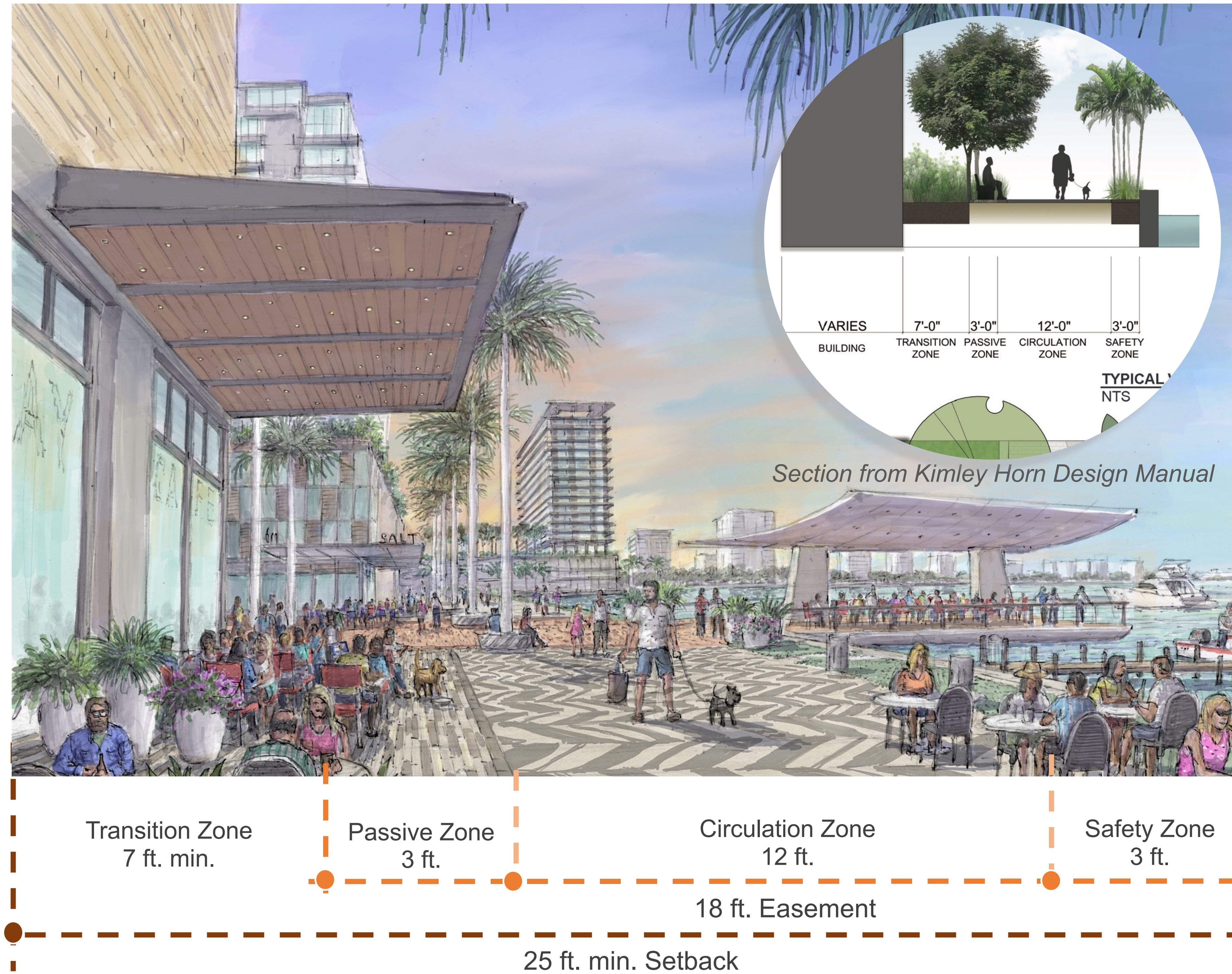
The overall easement is 18 ft. wide, set within a 25 ft. setback. The easement consists of three zones.

The **Circulation Zone** is in the center of the easement and is 12 ft wide (min.) To provide a memorable identity, it is paved with a distinctive design in the spirit of the patterns of Ipanema and Copacabana Beaches in Rio de Janeiro, which are recognized the world over. It may be used by walkers, joggers, and non-motorized, wheeled forms of transportation, such as bicycles, skateboard, rollerblades, and scooters. No electric motors are permitted.

Because railings and walls are not desirable along the water's edge, there is a 3 to 4-foot wide **Safety Zone**. It has planting beds or paving flush with the Circulation Zone. If paved, the paving will have a rougher texture to let pedestrians know they are approaching the edge. The cap of the seawall is 18-24 inches wide and 6 to 8 inches above the paving. Lighting in the Safety Zone will be provided by 8-inch diameter bollards that are 24 to 30 inches high and spaced 20 feet apart.

On the land side of the Circulation Zone, there is a **Passive Zone** that is at least 3 feet wide for benches, low planters, trees, light fixtures, and benches. Benches must face the water. It connects the Circulation Zone to the Transition Zone.

The remainder of the 25-foot setback is the **Transition Zone**. Though not technically part of the easement, it is continuous with the Island Walk and typically flush with it. The Transition Zone can be paved or landscaped, and it can be treated as an extension of the Passive Zone or the Circulation Zone. It should feature active frontages, ideally with restaurants and retail. It may be differentiated from the easement by its paving material, but this is not required.



Section from Kimley Horn Design Manual

Island Walk Illustrative Design

EMBRACE THE WATERFRONT

THE ISLAND WALK: SEAWALL PROMENADE

All the thought and planning behind the Island Walk and other related aspects of the NBV100 master plan will eventually come together to create a compelling **seawall promenade**, as imagined by an artist in the illustration on this page.

The Village has already received a grant to devise standards for the Island Walk based on the NBV100 vision. EAC consulting has been engaged to do this work as it works simultaneously on the seawall standards. With this design work in hand, the Village will be in a position to apply for much larger grants that would allow the Village to complete a large portion of the new Island Walk along the north side of Treasure Island in conjunction with replacement of the seawalls.

Existing condition along the northern edge of Treasure Island prevent the public from enjoying the water

A transformed waterfront featuring a seawall promenade

PROTECTED FROM WATER

Help mitigate climate change through resource conservation and reductions in carbon emissions. Prepare for sea level rise and severe weather events by improving stormwater management, constructing stronger seawalls, and raising public infrastructure.

Inconsistent seawall standards

Seawalls at different states of repair

Discontinuous Island Walk

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BUILD STRONG SEAWALLS

The entire Village is surrounded by seawalls, also known as bulkheads. They were constructed in the mid-twentieth century in the middle of Biscayne Bay before there was any land here. As the Bay was dredged to make the waters navigable, the spoils were placed inside the seawalls to create real estate.

As such, the seawalls were designed primarily to keep land in – not to keep water out. Most of them are aging – the life-expectancy is approximately thirty years – and most are not high enough for the levels of ever-rising king tides, let alone for storm surges. The vast majority are on private property. In short, most need to be repaired or replaced.

The Village is approaching the challenge of seawalls in two ways:

1) Devise common standards that will be written into the code. A new height for seawalls will be established that exceeds the current County minimum requirement of 5.0 ft.* This will be expressed as a minimum, not as a fixed height, so that property owners have the option to construct them taller in anticipation of rising sea levels. New seawalls should be structured to accommodate height extensions in the future. Property owners will be encouraged to use new building technologies such as fiber-reinforced concrete. As in Ft. Lauderdale, a date will be set (e.g., 2030) by which all seawalls must come into compliance, and this will be strictly enforced. The Village has hired EAC Consulting to devise new standards.

* See Note 2 on Seawall Comparison Table.

2) Devise a coordinated strategy for the replacement of seawalls so that large sections of seawall can be replaced in several large, coordinated efforts, one push for each zone or neighborhood, as logic dictates. The advantages are many. By representing multiple property owners together, the Village may be able to negotiate a lower price based on the quantity of work. Also, the Village may find a mechanism for financing the work and spreading out the cost over many years, or the Village may secure grants. These strategies could substantially reduce the costs for property owners compared to what they would pay individually. In addition, the final product will be more uniform, which will make it both stronger and more attractive.

Current Condition of Seawalls

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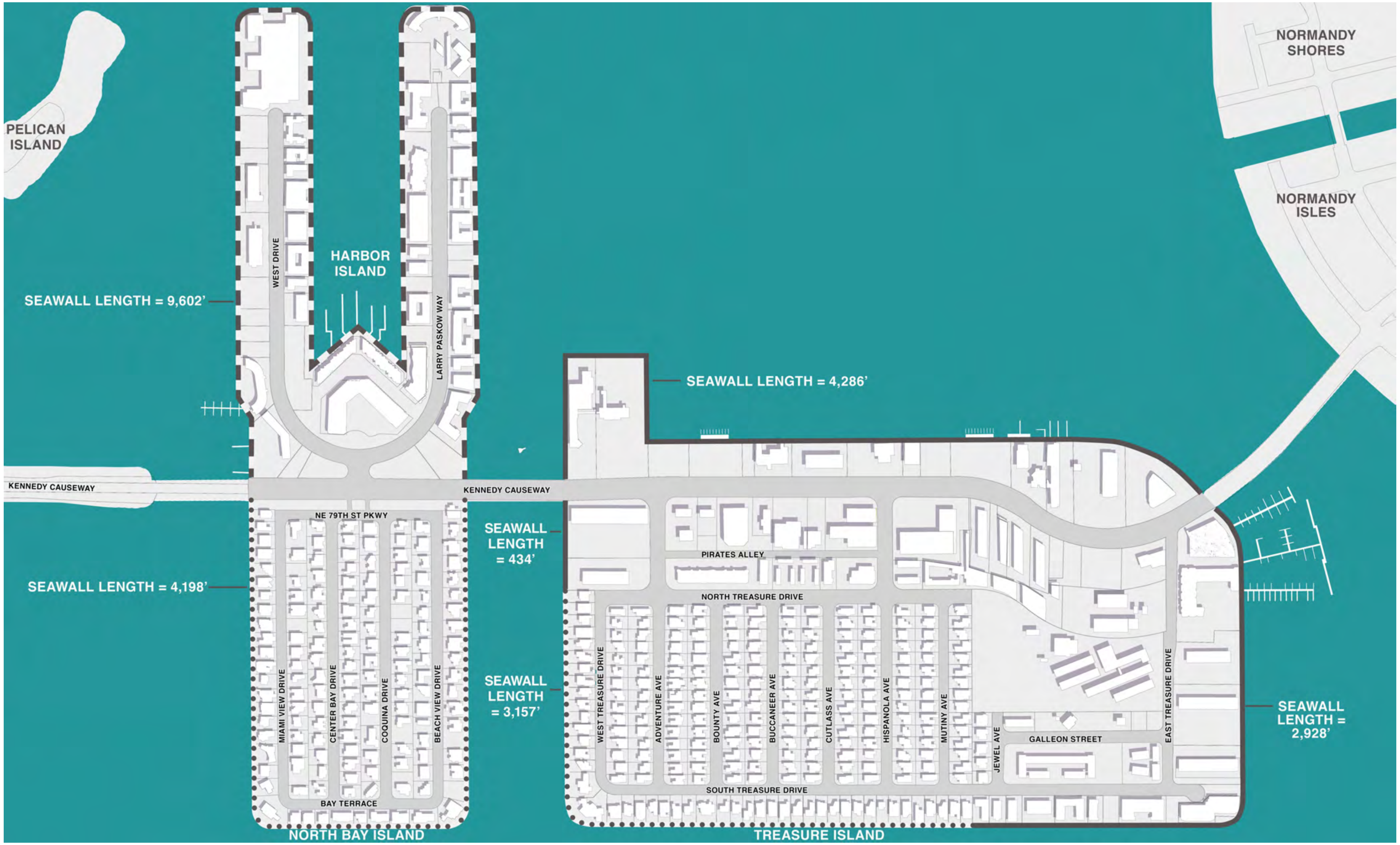
NBV SEAWALL PERIMETER ANALYSIS



Seawall (Mixed-use/Multi-family)

Seawall (Harbor Island)

Seawall (Single-family)

Seawall total length = 24,604' / 4.66 miles

NBV total land area = 0.37 SQ MI (0.96 km^2)

* Including the Causeway

This is a preliminary assessment undertaken by the DPZ team during the charrette of seawalls and their connection to the current distribution of density and intensity.

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SEAWALLS – RESILIENT STRUCTURE

Many of the original seawalls (or bulkheads) in North Bay Village follow a design that relies on tie rods. The tie rods connect to deadmen or some other counterweight underground to hold the panels in place and prevent them from overturning. This configuration is structurally efficient and made sense when the walls were first put in place before there was any fill on the landward side.

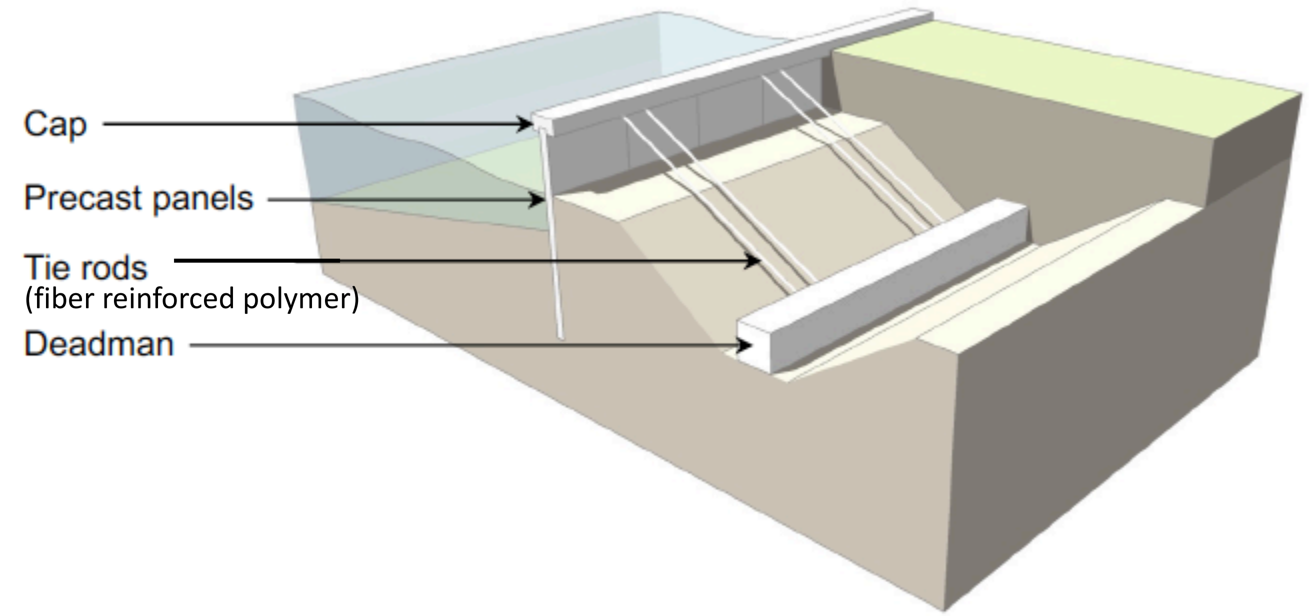
Illustrated here is an updated version that makes use of an innovative building material, namely non-corrosive fiber reinforced polymer materials pioneered by the Civil, Architectural, and Environmental Engineering Department at the University of Miami. One of the major advantages of this material is that it contains little or no steel, a material that is prone to rusting, especially in a salt-water environment. The Department generously provided engineered design drawings to NBV for use by residents seeking to replace their aging seawalls. This design served as the basis for the 3D illustrations by DPZ on the following pages.

EAC Consulting has been engaged by the Village to devise new seawall standards. They will provide several options, one of which will rely on tie rods and specify the aforementioned specialized material. However, it is worth noting that, over time, many tie rods have been severed to accommodate swimming pools, leading to the premature failure of these walls.

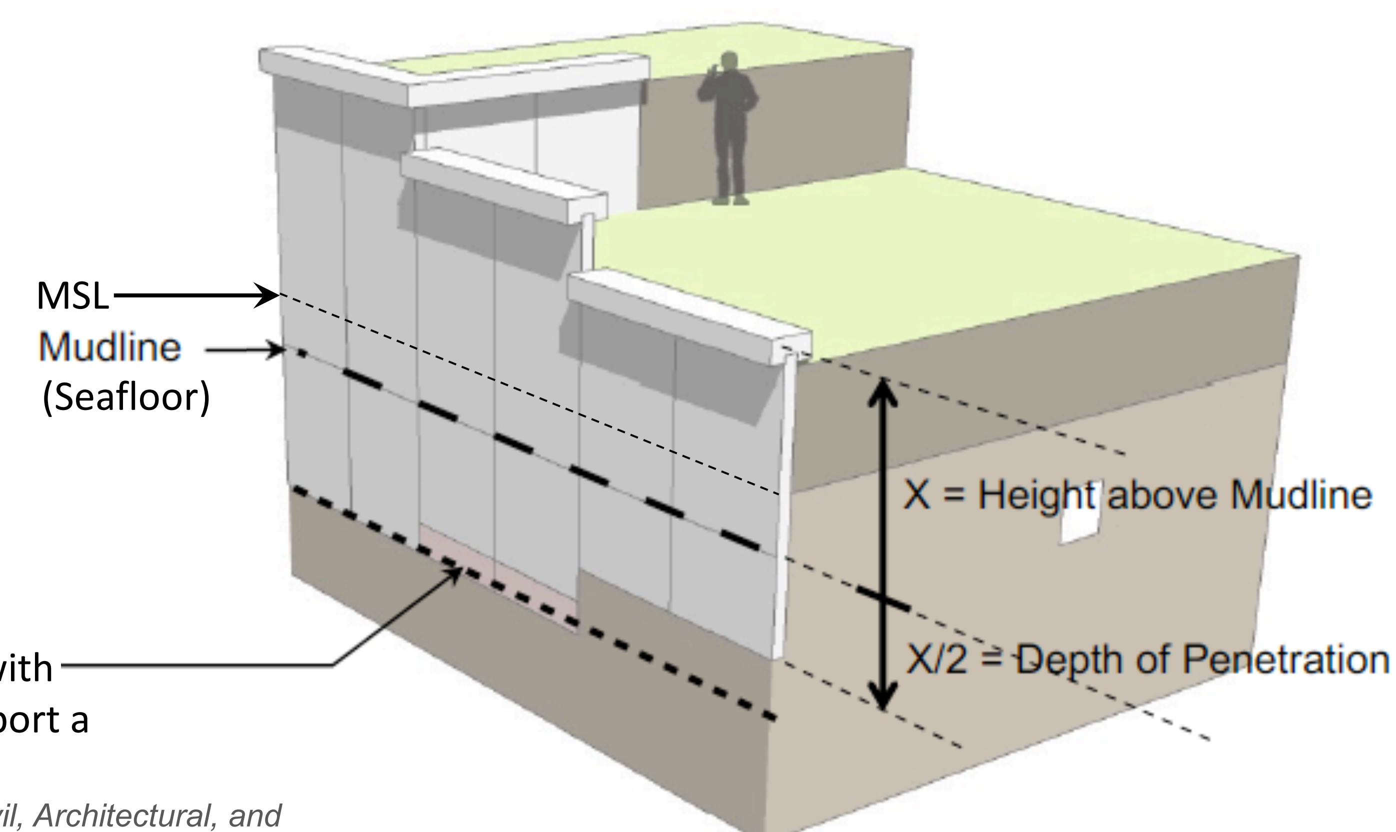
One alternative (not pictured) is a living seawall. This can take many forms, but one version consists of riprap placed in front of an aging seawall that needs repair or replacement. It is planted with mangroves and other native species. Living seawalls perform well in storm conditions and are easier to expand in the future than conventional seawalls. More stones are simply added to the top of the riprap, and the mangroves adjust their height naturally.

A living seawall typically involves the placement of riprap beyond the property line on submerged land in the riparian right-of-way. Unfortunately, the County does not at present allow property owners to encroach on submerged lands, which are technically owned by the State of Florida. However, individual property owners may elect to construct a living shoreline on their own property, as long as adequate height and protection to abutting properties are provided. EAC Consulting will provide standards for a range of options, including conventional approaches.

New seawalls must be built to a minimum height and be structured to accommodate later increases in height. The use of riprap will be encouraged wherever feasible, regardless of whether the riprap is planted as a living seawall.



Seawall with tie rods



Seawall foundations

New seawalls in NBV will be required to be structured to allow future increases in height.

3D illustrations by DPZ are based on designs by the Civil, Architectural, and Environmental Engineering Department at the University of Miami. These drawings are for illustrative purposes only. Dimensions and specifications are recommendations only. DPZ is not responsible for errors. EAC Consulting has been engaged by NBV to devise new standards for seawalls and the Island Walk. EAC Consulting is solely responsible for their final recommendations, which are pending.

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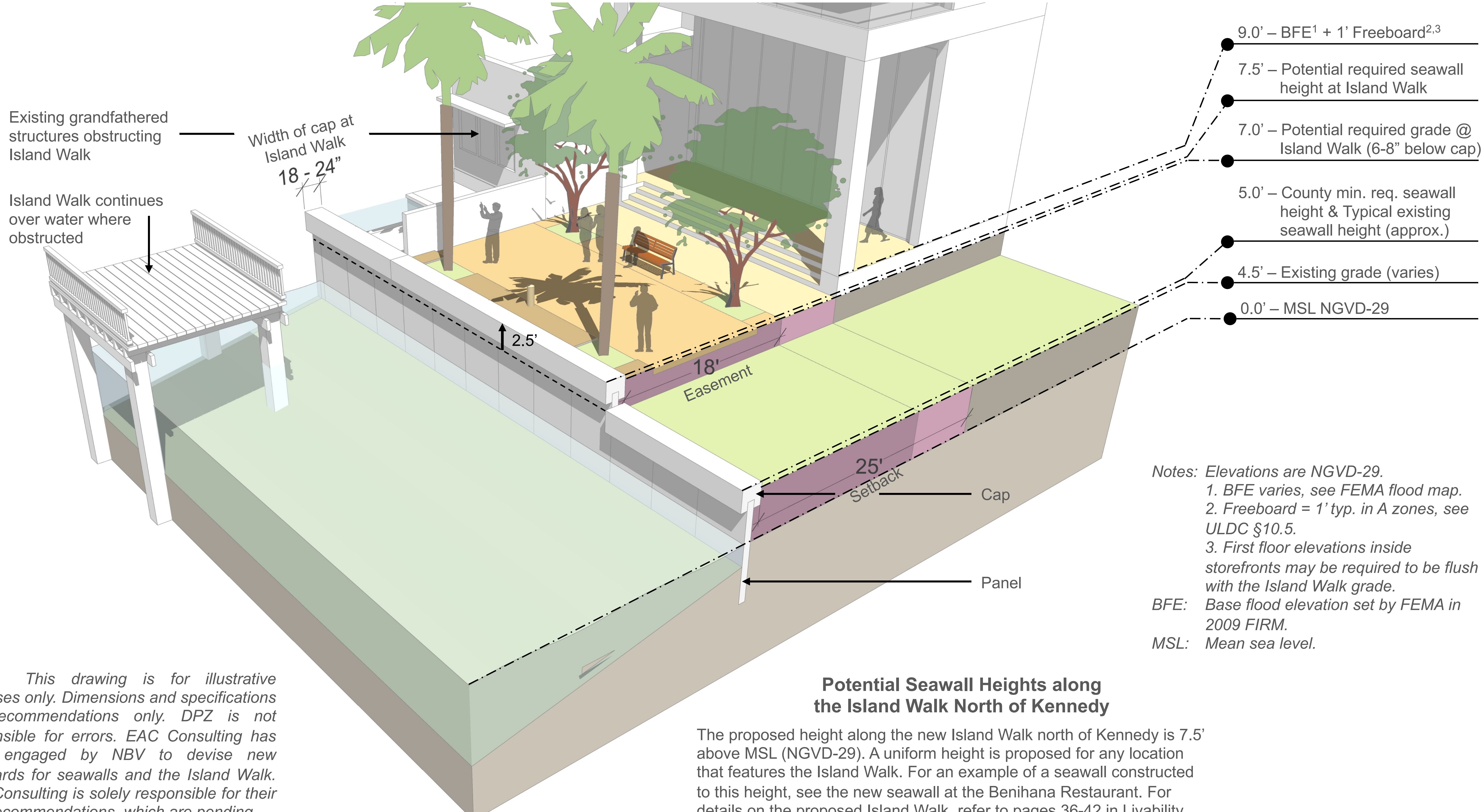
STRONG SEAWALLS AND ISLAND WALK NORTH OF KENNEDY



Notes: Elevations are NGVD-29.
1. BFE varies, see FEMA flood map.
2. Freeboard = 1' typ. in A zones, see ULDC §10.5.
3. First floor elevations inside storefronts may be required to be flush with the Island Walk grade.
BFE: Base flood elevation set by FEMA in 2009 FIRM.
MSL: Mean sea level.

Note: This drawing is for illustrative purposes only. Dimensions and specifications are recommendations only. DPZ is not responsible for errors. EAC Consulting has been engaged by NBV to devise new standards for seawalls and the Island Walk. EAC Consulting is solely responsible for their final recommendations, which are pending.

Potential Seawall Heights along the Island Walk North of Kennedy

The proposed height along the new Island Walk north of Kennedy is 7.5' above MSL (NGVD-29). A uniform height is proposed for any location that features the Island Walk. For an example of a seawall constructed to this height, see the new seawall at the Benihana Restaurant. For details on the proposed Island Walk, refer to pages 36-42 in Livability.

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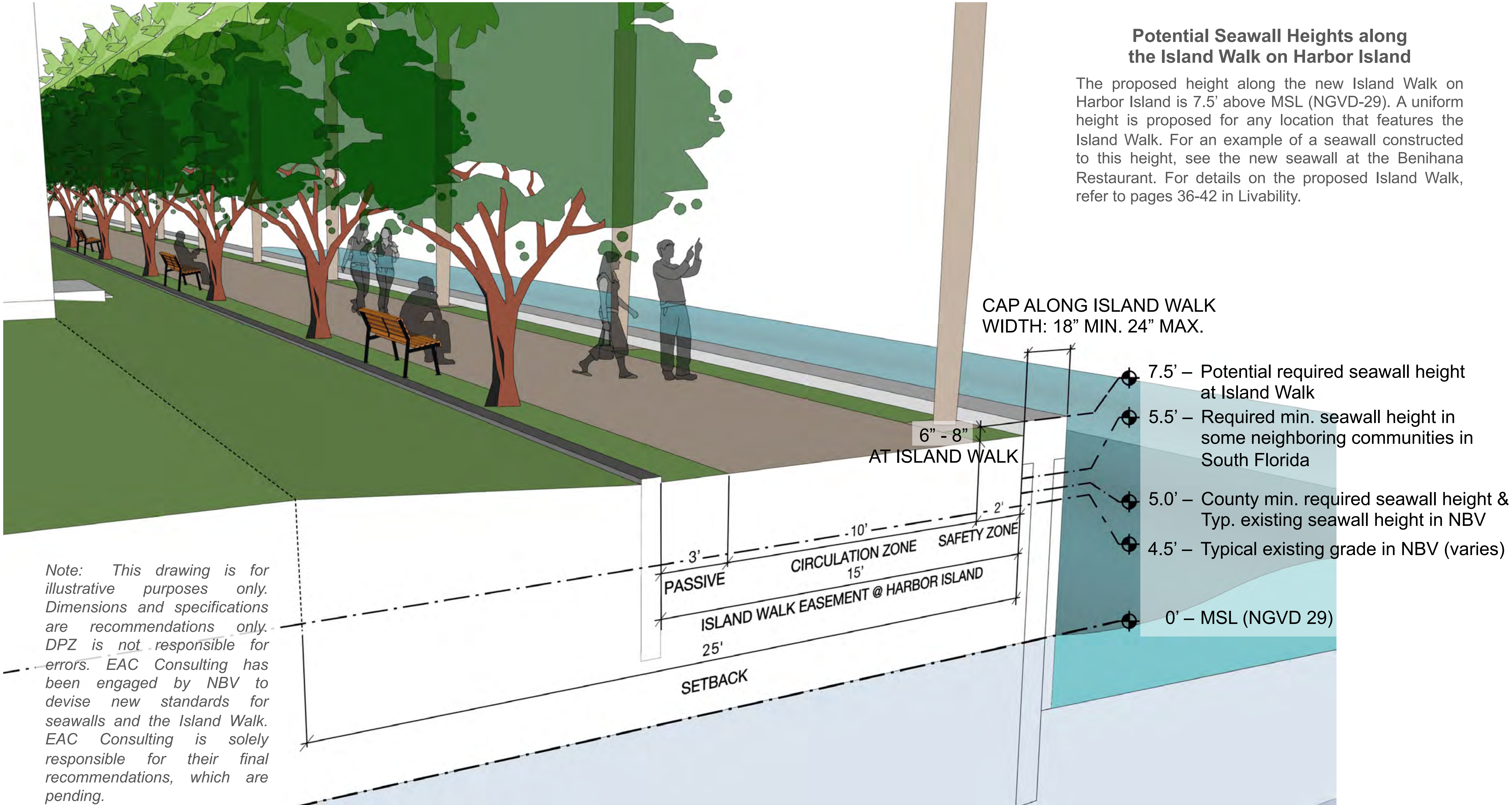
STRONG SEAWALLS AND ISLAND WALK ON HARBOR ISLAND



Potential Seawall Heights along the Island Walk on Harbor Island

The proposed height along the new Island Walk on Harbor Island is 7.5’ above MSL (NGVD-29). A uniform height is proposed for any location that features the Island Walk. For an example of a seawall constructed to this height, see the new seawall at the Benihana Restaurant. For details on the proposed Island Walk, refer to pages 36-42 in Livability.

Note: This drawing is for illustrative purposes only. Dimensions and specifications are recommendations only. DPZ is not responsible for errors. EAC Consulting has been engaged by NBV to devise new standards for seawalls and the Island Walk. EAC Consulting is solely responsible for their final recommendations, which are pending.

PROTECTED FROM WATER
RAISING PUBLIC INFRASTRUCTURE

NBV is already suffering from so-called nuisance flooding, which includes sunny day and king tide flooding, as illustrated in the photos on this page. (Undated images are available on the NBV website.)

The Miami-Dade County Office of Resilience has recommended that NBV elevate its roadways. Recognizing that this is an expensive and long-term project, the first step is to create a plan, which can be used to seek funding. In Miami Beach, this is part of their Stormwater Master Plan. Given that NBV will soon begin the process of creating its own Stormwater Management Plan and that the Village is currently devising the scope of work for this Plan, it is recommended that the Village incorporate a plan to elevate roadways into its new Stormwater Master Plan.

Though some places have raised their infrastructure high enough to remain dry during a major storm surge (e.g., Galveston, TX.), this is generally not considered practical in South Florida. However, it is practical to raise infrastructure high enough to reduce substantially or even eliminate nuisance flooding. This will be a boon to the local quality of life and help increase property values relative to other communities that have not solved such challenges. It will also lower ongoing maintenance costs. And it could buy precious time for evacuation and emergency access during a major storm.

Though resources are not currently in place to execute this project, putting a plan in place soon is important to the new form-based code and to facilitating development. For example, by establishing the benchmark “future crown of road”, it will make it clear for new projects where to set finish grade. The new form-based code will refer directly to this benchmark (or a similar benchmark.) This will allow redevelopment to harmonize over time, establishing a consistent elevation that anticipates future public improvements.

Creating a long-term plan will help NBV to prioritize infrastructure investments over the years. It may also help attract funding as a flagship project.

Adventure Avenue

North Treasure Drive

West Treasure Drive

West Treasure Drive

Appendix D

Background information from Univeristy of Miami

AASHTO Design Specifications for GFRP-RC Bridges: 2nd Edition.

M. Rossini[1], F. Matta[2], S. Nolan[3], W. Potter[3], A. Nanni[1]

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ABSTRACT: The development of a comprehensive bridge design national standard is paramount to allow for a wider and safe deployment of Glass Fiber Reinforced Polymer (GFRP) Reinforced Concrete (RC) in the transportation infrastructure. To respond to this demand, a task force of researchers, practitioners, and transportation officials lead by the University of Miami (UM), the University of South Carolina (USC), and the Florida Department of Transportation (FDOT), has developed a draft of the second edition of the Bridge Design Guide Specifications for GFRP-RC (BDGS-GFRP), now under consideration by the American Association of State Highway and Transportation Officials (AASHTO) committee T6. This paper deals with the salient contents of the document, with specific emphasis on the design of flexural members. Compared to the first edition, changes were proposed to reflect the state-of-the-art from archival literature and harmonize the design philosophy with that of other authoritative national and international standards.

KEYWORDS: GFRP-RC; design; guidelines; bridges; infrastructures.

1 INTRODUCTION

Fiber Reinforced Polymers (FRP) bars and strands are a viable corrosion-resistant solution for Reinforced Concrete (RC) and prestressed concrete (PC) in applications were corrosion of Mild Carbon Steel (MCS) and High Strength Carbon Steel (HSCS) represents a durability and safety concern. (Spadea et al., 2018). In particular, the application of Glass FRP (GFRP) bars is spreading, with a number of bridges built worldwide over the last 40 years (Bakis et al., 2002; Gooranorimi & Nanni, 2017). GFRP technology is tailored for application in aggressive environments. These include: coastal areas in sub-tropical environments (Nolan et al., 2018), cold weathered regions where de-icing salts are used and freeze-thaw cycles occur (Ahmad, 2003), urban and industrial areas where concrete is prone to carbonation and exposed to wet-dry cycles (Nanni et al., 2014), geotechnical applications where reinforcement is exposed to moist and contaminated soil (Mohamed & Benmokrane, 2014), and applications were the presence of stray currents may trigger corrosion in steel reinforcement (Spagnuolo et al., 2018).

Design principles for GFRP-RC are well established (Rossini et al., 2018a) and the technology is commercially available and spreading (Ruiz et al., 2017). Guidelines and regulations have been published in North America, Europe, Russia, and China (Rossini et al., 2018b). In the United States, design principles for GFRP-RC are detailed in guidelines issued by the American Concrete Institute (ACI, 2015). The deployment of GFRP-RC in buildings is regulated by the International Code Council (ICC) that maintains an Acceptance Criteria (AC) for GFRP bars (ICC, 2016). The deployment of GFRP-RC in infrastructural elements is regulated by the American Association of State Highway and Transportation Officials (AASHTO). AASHTO maintains a specific document that, in its first edition, only covers the design of GFRP-RC bridge decks and open-post railings (AASHTO, 2009). ASTM recently published standard specifications for GFRP bars (ASTM, 2017). The document does not hold binding status by itself, but it does once referenced in national design and construction codes and standards. The document is expected to relieve the need to include a chapter covering material specifications in design guidelines as it was done in the past. In Canada, the use of GFRP bars in buildings is covered by the guidelines issued by the Canadian Standardization Association (CSA, 2012). GFRP-RC deployment in infrastructures is regulated by the Canadian Highway Bridge Design Code (CHBDC) issued by CSA (2014). In Europe, guidelines for GFRP-RC design are published by the International Concrete Federation (*fib*, 2007). *fib* also includes GFRP-RC in its Model Code (*fib*, 2013). In Italy, guidelines for GFRP-RC design are published by the National Research Council (CNR, 2007). In Russia, the deployment of GFRP in buildings is regulated by a specific addendum to the national building code (Minstroy, 2018). The approach of the Russian building code is compatible with the one of the Eurocodes that, however, do not include GFRP-RC (CEN, 2005a). In China, the deployment of GFRP-RC in infrastructures is regulated by national guidelines (SAC, 2010).

The first generation of design guidelines and standards was issued in the late 90s and early 2000s. It succeeded in addressing the behavior of GFRP-RC structures, and the differences with respect to conventional steel RC members (Nanni, 1999). However, the limited experimental database available at that time called for the introduction of relatively severe safety factors (Jawaheri & Nanni, 2013).

The second generation of GFRP-RC design guidelines represents the recent state-of-the-practice. It expanded and refined the documents from the first-generation. However, little was done to address the issues that prevented one from taking full advantage of the efficiency and economical appeal of GFRP bars (Rossini et al., 2018a).

The third generation of design guidelines is currently under development and publication. It includes the 2nd edition of the AASHTO Bridge Design Guide Specifications for GFRP-RC Bridges (BDGS-GFRP) (AASHTO, 2018), the first edition of the ACI Building Code Provisions for Concrete Reinforced with GFRP Bars (currently under development), and an update of the CSA Canadian Highway Bridge Design Code (CHBDC) (scheduled for development).

2 RESEARCH SIGNIFICANCE

A draft of the second edition of AASHTO BDGS-GFRP (AASHTO, 2018) was developed by a task force of researchers, practitioners, and transportation officials led by the University of Miami (UM), the University of South Carolina (USC), and the Florida Department of Transportation (FDOT). Objectives included: updating the provisions to include state-of-the-art archival literature; making the provisions more rational and address the issues preventing one from taking full advantage of the mechanical and economic appeal of GFRP bars; making the design approach consistent with the AASHTO Bridge Design Specifications for traditional construction materials (BDS); and, harmonize the design philosophy with that of other authoritative national and international standards.

3 GUIDELINES INTEGRATION

Consistency and clarity in standards and guidelines are paramount to allow for safe and efficient design of structural members. At the same time, standardization is crucial to leverage deployment of innovative technologies in civil engineering. Nevertheless, GFRP-RC design guidelines typically exist as separate documents with respect to MCS-RC counterparts (AASHTO, 2009; 2017), or as addenda to national and local design codes (CSA, 2014). Furthermore, overlapping exists, and FRP-RC/PC design guidelines have different approaches one with respect to the other and with respect to design guidelines for traditional structural materials (Rossini et al., 2018b). Differences include: the definition of the material properties to be used for design purposes; the structure of the design equations; and, the value and definition of the design parameters to be used in these equations.

The ideal setting to leverage wider deployment of GFRP-RC in substantial applications entails embedding GFRP bars as an alternative reinforcement solution in a comprehensive standard (Nolan & Nanni, 2017). The approach can be expanded to include other materials, as well as PC applications (Rossini et al., 2018b).

4 DESIGN APPROACH

Rossini et al. (2018b) outlined a unified design approach to FRP-RC/PC. The approach was validated on an FRP-RC/PC pedestrian bridge reinforced with Glass FRP bars, Basalt FRP bars, and Carbon FRP strands. The approach served as a framework for developing the draft of the second edition of AASHTO BDGS-GFRP and is summarized in the following with specific reference to the case of GFRP-RC.

Any mechanical problem can be defined as a system of equilibrium, compatibility and constitutive equations. Structural theories introduce assumptions to simplify the mathematical formulation of common mechanical problems, like the beam model. Classical Euler-Bernoulli assumptions hold valid in GFRP-RC bended elements, and sectional analysis can be carried out. Rigorously, the only difference with respect to MCS-RC is in the constitutive law used to model the reinforcing bars. Similarly to the design of MCS-RC, limitations in the exploitability of the materials, amount of reinforcement, and maximum strains and deflection are introduced to ensure structural assumptions are met and the desired level of safety and reliability is provided.

4.1 *Material properties*

GFRP is a brittle composite material, elastic until failure, stronger, but less stiff with respect to MCS. The guaranteed strength (f_{fu}^*) of a GFRP bar is defined as the experimental average value minus three standard deviation (ACI, 2015), corresponding to the 99.9th strength percentile. The definition is reported in Equation 1 for clarity. The approach is more conservative with respect to the calculation of characteristic strengths for steel reinforcement and concrete, traditionally defined as the average value minus 1.64 standard deviation – 95th strength percentile – under the assumption of normal distribution (CEN, 2005a).

$$f_{fu}^* = f_{fm} - 3\sigma_f \tag{1}$$

The strength of commercially available GFRP bars can vary from product to product at varying fiber content and manufacturing techniques (Ruiz et a., 2017). At the time of design, the bar manufacturer is typically not defined. Thus, the minimum guaranteed strength required for certification per ASTM D7957 (ASTM, 2017) is taken as the specified tensile strength for design purposes in spite of an experimental value. The specified strength ($f_{fu}^{'}$) is always less than or equal than the guaranteed experimental strength (f_{fu}^*) of the specific batch of bars that will be deployed in construction, as shown in Equation 2.

$$f_{fu}^{'} \leq f_{fu}^* \tag{2}$$

Tracing a straight line to limit the exploitability of different products and material systems may slow down the growth of the GFRP industry. Nevertheless, the need for standardization is paramount. A possible solution may lay in the definition of different strength grades, as traditionally done for MCS bars (ASTM, 2016), steel profiles (AISC, 2017; CEN, 2005b), and concrete (FDOT, 2018; CEN, 2005a).

FRP composites are known to experience strength degradation following long-term exposure to the environment (ACI, 2015; *fib*, 2007). To account for the phenomenon, the design strength (f_{fd}) of the material is defined per Equation 3 including an environmental reduction factor (C_E). The approach is in line with the principles of ACI (2015).

$$f_{fd} = C_E f_{fu}^{'} \tag{3}$$

The design strength of the material is the reference value for design calculations, both at the ultimate limit state (ULS) and service limit state (SLS). Furthermore, the strength of FRP under sustained load is reduced to avoid creep rupture (ACI, 2015; *fib*, 2007). Resorting to the nomenclature suggested by Rossini et al. (2018a), a creep rupture reduction factor (C_c) is applied to the design strength in order to define the design strength against creep rupture under sustained load ($f_{f,c}$) as in Equation 4. Similarly, a fatigue reduction factor (C_f) is applied to the design strength in order to define the design strength under cyclic loading ($f_{f,f}$) as in Equation 5.

$$f_{f,c} = C_c f_{fd} = C_E C_c f_{fu}^{'} \tag{4}$$

$$f_{f,f} = C_f f_{fd} = C_E C_f f_{fu}^{'} \tag{5}$$

The brittle nature of FRP reinforcement implies the possibility to either have over-reinforced flexural members that may fail because of concrete collapse in the compression zone, or under-reinforced flexural members that may fail because of reinforcement rupture in the tension zone (ACI, 2015). The two failure modes are characterized by two different strength reduction factors – ϕ_c and ϕ_t respectively – defined to guarantee the same level of safety in the two cases. A flexural member can also undergo shear failure. In this case the strength reduction factor ϕ_s is aligned to values prescribed for MCS-RC in ACI (2014).

GFRP bars lack the plastic plafond typical of MCS bars. Thus, GFRP-RC flexural members do not feature ductile behavior at failure. Nevertheless, GFRP bars reach strain levels higher than the 0.005 ductility threshold set for MCS in ACI (2014). Thus, GFRP-RC flexural members feature a pseudo-ductile behavior comparable to what is required of MCS-RC to foresee upcoming failure. Figure 1a compares the mechanical behavior of GFRP and MCS bars M13. Figure 1b compares the flexural strength reduction factor proposed for GFRP bars to traditional values used for MCS (AASHTO, 2017). The strength reduction factors are plotted as a function of the strain reached by the reinforcement at sectional failure. The diagram is adapted from AASHTO (2017).

The different bond characteristics of GFRP bars with respect to steel reinforcement is accounted for introducing a bond reduction factor (C_b). The parameter is defined in Equation 6 as the inverse of the bond reduction coefficient (k_b) as defined in ACI 440.1R (ACI, 2015). By this definition, the bond reduction factor increases at increasing performances, consistently with the other design factors. Better bond performances enhance crack control and reduce crack width at equal load level (ACI, 2015).

$$C_b = 1/k_b \tag{6}$$

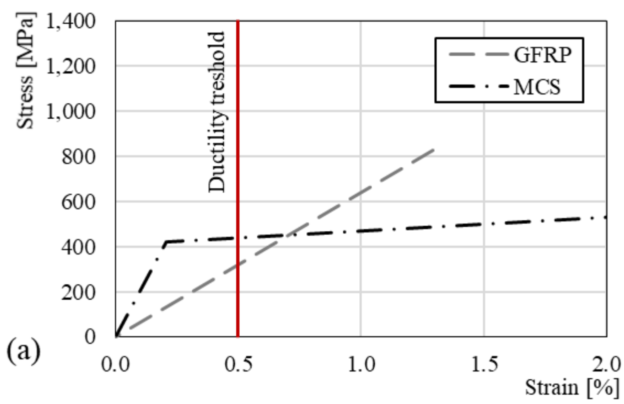
4.2 *Design factors.*

Table 1 provides a summary of design factors as reported by international design guidelines, along with the values adopted in the second edition of AASHTO BDGS-GFRP. The flexural strength reduction factor for compression-controlled failures (ϕ_c) is raised from 0.65 to 0.75 (+15%) with respect to the first edition of AASHTO BDGS-GFRP (AASHTO, 2009). The value is in line with findings of Jawaheri & Nanni (2013). The creep rupture reduction factor (C_c) is raised from 0.20 to 0.30 (+50%) with respect to the first edition of AASHTO BDGS-GFRP. The value is more reflective of the performances of ASTM-compliant GFRP bars, and about 50% of the experimental findings of Perigny et al. (2012), Sayed-Ahmed et al. (2017), and Keller et al. (2017). Similarly, the fatigue reduction factor (C_f) is raised to 0.25 (+25%) for alignment with international standards (CNR, 2007; *fib*, 2013, CSA, 2014). The bond reduction factor (C_b) is raised from 0.71 to 0.83 (+17%). The value is reflective of the good bond performances of GFRP bars (Gooranorimi et al., 2018) and is more conservative with respect to international guidelines (CSA, 2017).

Table 1. Design parameters for GFRP-RC

	CNR 2007	*fib* 2013	CSA 2014	ACI 2015	AASHTO 2009	AASHTO 2018
ϕ_c	0.67	0.67	0.75	0.65	0.65	**0.75**
ϕ_t	0.60	0.80	0.55	0.55	0.55	**0.55**
ϕ_s	-	-	-	0.75	0.75	**0.75**
C_E	0.70	0.55 (1)	1.00	0.70	0.70	**0.70**
C_c	0.30	0.30	0.25	0.20	0.20	**0.30**
C_f	0.30	0.50	0.25	0.20	0.20	**0.25**
C_b	0.59	0.71 (1)	1.00	0.71	0.71	**0.83**

(1) from *fib* bulletin 40 (*fib*, 2007).



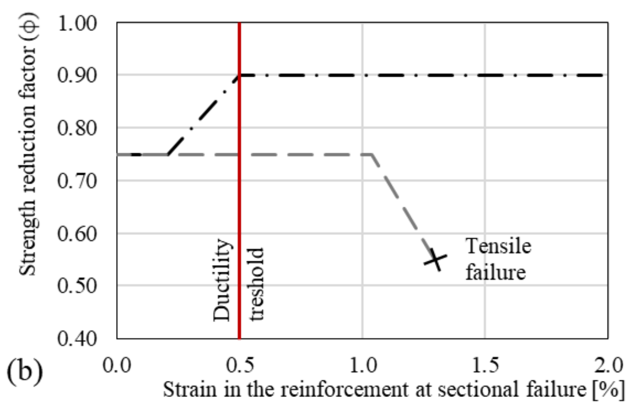


Figure 1. Mechanical properties (a) and flexural strength reduction factors (b) for M25 bars made with GFRP and MCS.

4.3 *Limit states.*

As for the case of MCS-RC, a GFRP-RC flexural member must be designed against a number of Ultimate Limit States (ULSs) and Service Limit States (SLSs). ULSs include compression failure of the concrete or tension failure of the GFRP bars under factored load. Furthermore, GFRP bars can experience creep rupture under sustained load, and fatigue rupture under cyclic load. These conditions are verified under service loads but represent ULSs in the sense that failure to comply may result in the catastrophic collapse of the member. SLSs include a limit on deflection (L/800 for vehicular bridges), a limit on crack width (0.7 mm), and a limit on concrete stresses under sustained load (0.45 f_c'). The relatively low stiffness of GFRP bars may result in SLSs governing the design.

According to ACI (2015) the creep rupture limit state must be verified under sustained load. The AASHTO BDS for traditional construction materials lacks the explicit definition of a sustained service load. Thus, the first edition of the AASHTO BDGS-GFRP considered the entire amount of service load as sustained. The assumption is overconservative and not aligned with international bridge design guidelines (CEN, 2005a). In the second edition of AASHTO BDGS-GFRP the sustained portion of the service load is set equal to the dead load (DL) plus 20% of the live load (LL) as shown in Equation 7. The approach is in line with ACI (2015) and more conservative with respect to international guidelines that only consider DL as sustained (CEN, 2005a; CNR, 2007).

$$Sustained\ Load = DL + 0.20\,LL \qquad (7)$$

According to ACI (2015) the fatigue rupture limit state must be verified under the sum of the sustained load plus the maximum load experience in a fatigue cycle. In translating this provision to AASHTO language, the total fatigue load is defined as the sum of the Dead Load (DL) plus the factored transient loads defined per AASHTO (2017) Fatigue load combination Fatigue I (F1). The fatigue load combination is reported in Equation 8. The issue of load combinations for creep rupture and cyclic fatigue is also discussed by Rossini et al. (2018a).

$$Fatigue\ Load = DL + F1 \qquad (8)$$

4.4 *Philosophy and applicability.*

The RC and PC design section of the AASHTO BDS has recently underwent a major update (Montgomery et al., 2017). The second edition of the AASHTO BDGS-GFRP is compatible with the most recent edition of the AASHTO BDS (AASHTO, 2017). The GFRP counterpart reflects the structure and organization of the main document and minimizes the differences in design equations to ease application by practitioners. Differences are limited to adjusting design parameters and material properties to account for the different behavior of GFRP bars with respect to MCS. Furthermore, the second edition of the AASHTO BDGS-GFRP is meant for application along with the material specifications published by ASTM (2017). This sets the first example for the next generation of integrated GFRP-RC design, construction, and material guidelines to be consistently developed without overlapping.

The major limitation of the first edition of the AASHTO BDGS-GFRP (AASHTO, 2009) laid in the limited field of application as it only covered bridge decks and open-post railings. The second edition of the AASHTO BDGS-GFRP covers all the members that compose a RC bridge. This includes decks, girders, bent caps, bulkhead caps, bearing piles, sheet piles, gravity walls, open-post railings, continuous railings, and approach slabs. It is the first regulation to cover GFRP-RC substructure, and is the most complete guideline for GFRP-RC design. Its provisions have been developed and tested on a number of structures currently built or under construction. This includes the Innovation Bridge discussed by Rossini et al. (2018b), and the Halls River Bridge discussed by Rossini et al. (2018a).

5 PARAMETRIC ANALYSIS

In developing the draft for the second edition of the AASHTO BDGS-GFRP, parametric analysis was used as a tool to quantify the effect of the proposed variation in the design parameters. In the following, a selection of the results of the parametric analysis is discussed. The methodology adopted will be briefly summarized. For more details, reference is made to Rossini et al., (2018a).

The study focuses on the GFRP-RC pile cap of the Halls River Bridge currently under construction in Homosassa, FL (Rossini et al., 2018a) (Figure 2). The element is deemed representative of large under-reinforced

GFRP-RC members acting as pile caps in short-spanned traffic bridges. Given their exposure condition and proximity to water surfaces, these members represent typical applications for GFRP-RC and are of particular interest for FDOT.

Each parametric curve is constructed by calculating the minimum number of M25 GFRP bars that satisfies the specific design requirement: moment capacity, minimum reinforcement, creep rupture, fatigue, and crack width limits. The design for positive moment capacity based on ACI (2015) and AASHTO (2009) resulted in 16 M25 bars with a guaranteed strength of 550 MPa and an elastic modulus of 45 GPa, for a total area of 8084 mm^2 to resist a factored moment demand of 575 kN-m. The design for positive moment capacity based on the second edition of AASHTO BDGS-GFRP resulted in 9 M25 bars, for a total area of 4547 mm^2. This corresponds to a reduction of 40% with respect to the first edition (Rossini et al., 2018b).

Figure 3 shows the influence of the variation of a selection of parameters on the required amount of reinforcement. The design demand is represented in terms of required number of longitudinal M25 bars. For each diagram, design equations are plotted as a function of the selected range. The remaining parameters are set constant and equal to values recommended in the first and second edition of the AASHTO BDGS-GFRP respectively. The results presented in Figure 3 are case-dependent, but the trend of the curves is indicative in general.

Comparing Figure 3a and Figure 3b shows how the rationalization of the sustained and cyclic load demand discussed in section 4.3 reduces the influence of the cyclic fatigue and creep rupture requirements from governing to negligible. Relaxation of the creep rupture reduction factor (C_c) from 0.20 to 0.30, and the fatigue reduction factor (C_f) from 0.20 to 0.25 contributes to this outcome, but the effect is limited as shown in Figure 3b.

Experimental results suggest that further margin for improvement exists, but additional research is required. The prioritization of research into creep rupture and cyclic fatigue endurance limits is suggested by the limited database available and by mechanical considerations discussed by Rossini et al. (2018a).

The crack width requirement governs over the strength requirement as show in Figure 3c and Figure 3d. A relaxation of the bond reduction factor (C_b) from 0.71 to 0.83, along with a relaxation of the crack width limit (w) from 0.5 mm to 0.7 mm, and a relaxation of the minimum concrete clear cover (c_c) from 51 mm to 38 mm, reduces the required amount of reinforcement to fulfill SLSs by about 30%.

The design of large section MCS-RC members is typically governed by minimum reinforcement considerations. This is not the case for GFRP-RC members designed according to the first edition of AASHTO BDGS-GFRP. Conversely, the minimum reinforcement requirement governs the design of GFRP-RC large section members according to the draft of the second edition of AASHTO BDGS-GFRP. This follows the rationalization of the sustained and cyclic load demand discussed in section 4.3. Furthermore, the minimum reinforcement requirement has been aligned to the formulation adopted in AASHTO BDS (2017). This approach ensures a minimum level of strength and ductility to the system as a function of the mechanical properties of the reinforcement, and offsets overconservativeness in some cases. Details are discussed by Rossini et al. (2018a; 2018b).

Figure 3e and Figure 3f show how improving the stiffness (E_f) – and therefore the strength (f_{fu}^*) – of GFRP bars amplifies the benefits of the proposed refinements in design limits. However, the increment in the elastic modulus should not come from a mere increase of the effective cross-sectional area compared to the nominal design area, but rather a combination of increased fiber ratio, improved material properties, and superior manufacturing quality control.

An improved quality control of the product may help refining most of the design parameters discussed but needs to be reflected in more performing material specifications (ASTM, 2017) before designers can take full advantage of it.

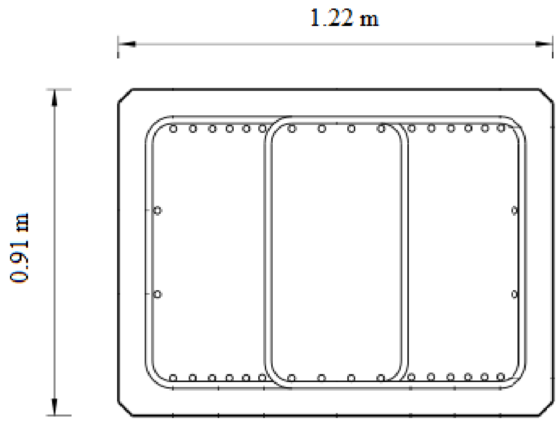


Figure 2. Transversal section of the pile cap of the Halls River Bridge.

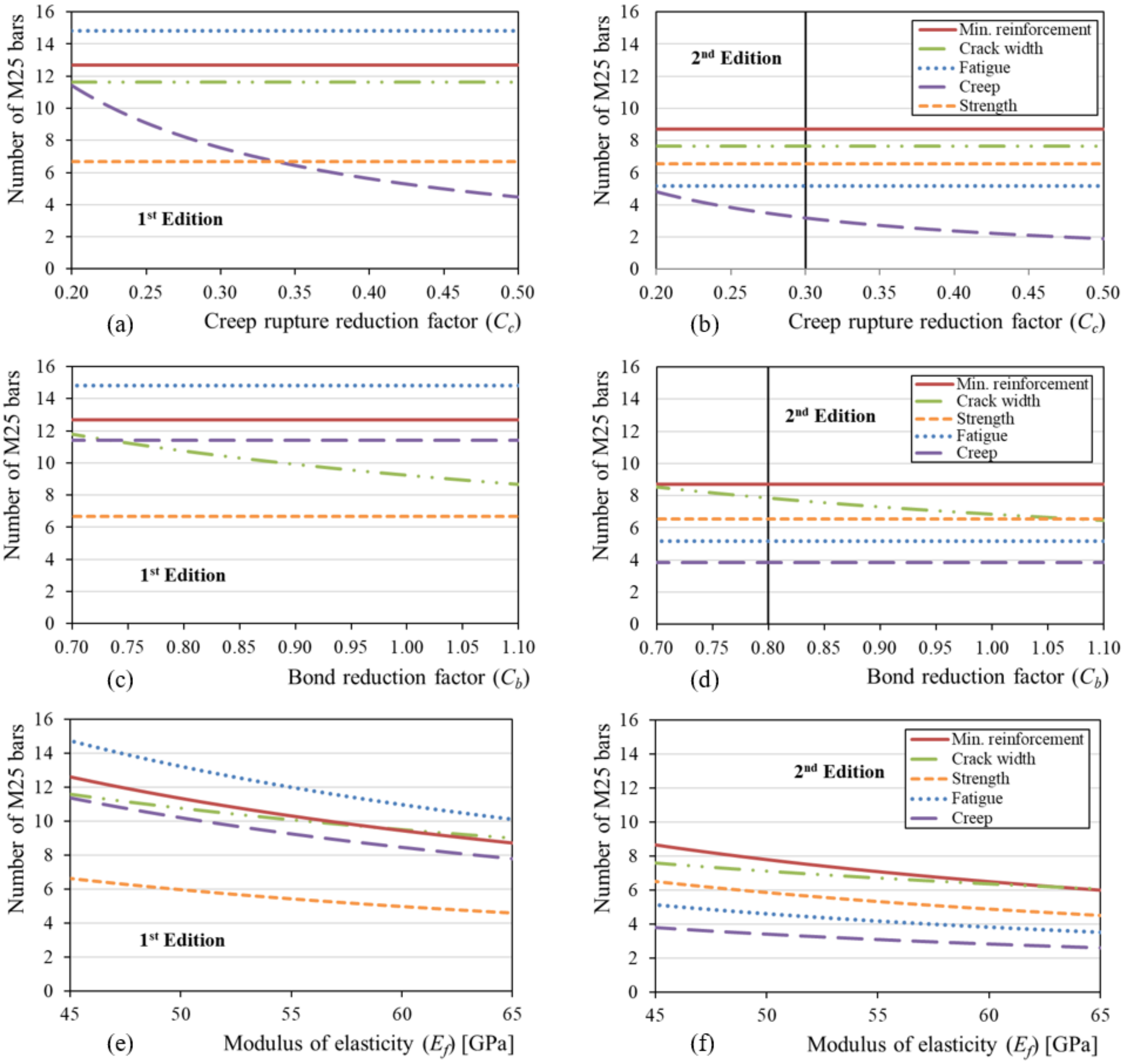


Figure 3. Required number of M25 bars as a function of the variation of the design parameters using the equations of AASHTO BDGS-GFRP 1st edition (left) and AASHTO BDGS-GFRP 2nd edition (right).

6 CONCLUSIONS

In this study the salient contents of the draft of the second editions of the AASHTO BDGS-GFRP are discussed along with the conceptual framework functional to the development of the document. Specific emphasis is devoted to flexural members. The differences with respect to the first edition are quantified resorting to parametric analysis.

The second edition of AASHTO BDGS-GFRP (AASHTO, 2018) aims to provide a rational and consistent framework for the design of GFRP-RC bridge structures. This is expected to raise awareness and leverage wider deployment of non-corrosive reinforcement solutions in infrastructures. Furthermore, the definition of a consistent regulatory framework is expected to help define and prioritize Research and Development (R&D) areas – at the academic, private, and regulatory level – to make the technology more efficient, economical and environmentally appealing (Rossini et al., 2018a). Specific features of the document include:

1. Design parameters and procedures have been updated to reflect advancements in the state-of-the-art. This includes refinement of the strength reduction factor for compression-controlled failures (ϕ_c); refinement of the creep rupture reduction factor (C_c), fatigue reduction factor (C_f), and bond reduction factor (C_b).

2. Design demands and limit states have been made more rational and consistent with national and international guidelines (CEN, 2005a; CNR, 2007; ACI, 2015; AASHTO, 2017). This offsets overconservativeness in creep rupture and cyclic fatigue demands.
3. Design equations have been updated to align the document to the most recent edition of the AASHTO Bridge Design Specifications (AASHTO, 2017). This creates a familiar environment for the practitioners approaching GFRP-RC for the first time and resolve some inconsistencies.
4. The document is expanded to include all the reinforced concrete components of a bridge structure. The first edition only included bridge decks and open-post railings. It is the first guideline to include provisions for GFRP-RC substructures.
5. The document is structured to automatically benefit from any refinement in material specifications issued by ASTM (2017). This would not be the case if an additional material specification chapter was to be introduced as done in the first edition.

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SEACON and Resilient FRP-RC/PC Solutions: The Halls River Bridge.

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ABSTRACT: the SEACON project features the University of Miami (UM), the Florida Department of Transportation (FDOT), along with European partners (Politecnico di Milano) and representative of the industrial sector working toward the development of innovative material solutions to address sustainability and resilience challenges in construction. The project aims to develop sustainable concrete solutions using seawater and chloride-contaminated aggregates. An integral component involves validating Fiber Reinforced Polymers (FRP) and stainless steel (SS) as non-corrosive reinforcement for Reinforced Concrete (RC) and Prestressed Concrete (PC) applications. The Halls River Bridge (HRB) features large-scale implementation of innovative materials. It showcases the SEACON research outcomes and serves as a proof-of-concept for the validation of design philosophies to be included in the new generation of FRP design guidelines. This paper speaks about the issues of design of a non-corrosive FRP-RC/PC structure. HRB is presented as a successful case study.

KEYWORDS: FRP-RC; FRP-PC; design guidelines; resilience; sustainability.

1 RESEARCH SIGNIFICANCE

Corrosion of Mild Carbon Steel (MCS) and High-Strength Carbon Steel (HSCS) reinforcement is a durability concern in aged Reinforced Concrete (RC) and Prestressed Concrete (PC) structures. Extensive development of transportation infrastructure, in combination with aggressive environments, may represents a severe Maintenance, Rehabilitation and Replacement (MRR) liability both in the United States (Nolan & Nanni, 2017) and Europe (Bertolini et al., 2005). RC corrosion is ubiquitous, but greatly exacerbated by aggressive sub-tropical environments (Nolan et al., 2018) and exposure to de-icing salt and carbonation in cold-weathered regions (Ahmad, 2003).

In the case of transportation infrastructure in coastal areas, the immediate corrosion problems are experienced by bridge substructures, sheet pile bulkheads, pile bent caps, bearing piles, and seawalls (Rossini et al., 2018b). In the State of Florida alone, approximately 3,600 coastal miles are armored with aging sheet piles for an estimated $21B MRR liability (Nolan et al., 2018).

A traditional solution to address reinforcement corrosion entails designing concrete mixes with limited permeability and superior durability characteristics (Bertolini et al., 2005). This approach lays the durability burden on cement manufacturer and ready-mix producer. Furthermore, it typically involves cement-rich concrete mix-designs. Whereas low-impact cements are being investigated (Bertola et al., 2018), the raising demand for high-quality mixes may exacerbate the environmental impact of the concrete industry (Bertola et al., 2016).

Fiber-Reinforced Polymers (FRP) represent a proven non-corrosive alternative to steel in rehabilitation (Rossini et al., 2018c) and new constructions (Spadea et al., 2018). Commercially available solutions include Glass FRP (GFRP) bars, Carbon FRP (CFRP) strands, and Basalt FRP (BFRP) bars (Ruiz et al., 2017). Deployment of FRP reinforced concrete (FRP-RC) and FRP prestressed concrete (FRP-PC) eliminates the issue of reinforcement corrosion, irrespectively of the concrete mix-design (Khatib et al., 2017).

The SEACON-Infravation project is a research effort jointly funded by the European Commission and government agencies such as the US Federal HighWay Administration (FHWA) and the Italian ANAS. SEACON aims to develop low-impact concrete using seawater, chloride-contaminated cement, and recycled aggregates. Sea-concrete requires coupling with non-corrosive reinforcement. Thus, within SEACON, lays the validation, deployment and support for standardization of FRP, MSS, and HSSS reinforcement.

2 HALLS RIVER BRIDGE

The Halls River Bridge (HRB) is a short-spanned traffic bridge currently under construction in Homosassa, Florida (Figure 1a). The bridge is part of a replacement project for an existing structure that reached functional deficiency and is aged beyond its service life (Figure 1b). The new structure comprises five spans for a total length of 56.6 m. It serves as the only passageway over the Halls River for the community of Homosassa Springs. The water way is tidally affected by seawater contamination, particularly during storms, given the proximity to the Gulf of Mexico.

Given its exposure conditions and structural configuration, the HRB was selected to serve as demonstrator for both the SEACON project and the Florida Department of Transportation (FDOT) Transport Innovation Challenge (TIC). One of the aims of the latter is to leverage the deployment of non-corrosive technologies in transportation infrastructure. Furthermore, the HRB served as test-bed and active laboratory for the development of a new generation of FRP design and construction guidelines. This includes the new edition of the Bridge Design Guide Specifications for GFRP-RC (BDGS-GFRP) (AASHTO, 2018). The draft of the AASHTO BDGS-GFRP was developed by a task force lead by the University of Miami (UM) and representatives of State Departments of Transportation (DOTs) and FHWA.

The HRB comprises a number of innovative material and structural solutions targeting a reduced environmental impact and an extended service life of 100+ year (Cadenazzi et al., 2018a). The structure includes CFRP-PC bearing piles, CFRP-PC/GFRP-RC sheet piles, hybrid HSCS-PC/GFRP-RC sheet piles, GFRP-RC pile bent caps and bulkhead caps, a GFRP-RC bridge deck, GFRP-RC traffic railings, GFRP-RC approach slabs and a 20 m long GFRP-RC gravity wall. The original design implemented Hillman Composite Beams (HCB), consisting of a composite GFRP shell encasing a steel-reinforced concrete shallow tied-arch and lightweight filling foam. This complex structural solution was developed under the National Cooperative Highway Research Program's Innovations Deserving Exploratory Analysis (NCHRP-IDEA) program and selected by FDOT for further exploration (Cadenazzi et al. 2018b). An alternative GFRP-RC solution that provides equivalent strength and performance is shown in this study.

In addition to innovative reinforcement solutions, the HRB features deployment of sustainable concrete mixes in the elements of the substructure. Concrete mixed with seawater is used for the bulkhead cap, concrete with Recycled Concrete Aggregates (RCA) and concrete with Recycled Asphalt Pavement (RAP) aggregates is used for the GFRP-RC gravity walls. White cement concrete and another mixture of high-content slag and fly ash are used in the GFRP-RC traffic railings for investigation of enhanced night-time and wet weather visibility. The bulkhead cap includes test-blocks on the water side to be periodically extracted and tested to assess FRP durability in chloride-exposed sea-concrete. The test blocks include GFRP, CFRP, and BFRP bars.

Figure 1. Halls River Bridge under construction (a) and existing bridge before demolition (b). [Courtesy of Astaldi Construction Corporation]

3 DESIGN CONSIDERATION

3.1 *Guidelines and design approach*

A number of design and construction guidelines for FRP-RC and FRP-PC exist worldwide. A detailed discussion of CFRP-PC design can be found in Spadea et al. (2018). Principles of GFRP-RC design are discussed by Rossini et al. (2018a).

In general, the brittle nature of FRP is addressed by introducing a specific reduction factor for the flexural strength of the structure (ϕ). The ultimate strength of the material is reduced accounting for the effects of long-term exposure to moisture by introducing an environmental reduction factor (C_E). The jacking stress for prestress is limited to safe values by introducing a jacking reduction factor (C_j). The resistance to sustained stress is reduced to avoid creep rupture by introducing a creep rupture reduction factor (C_c). The difference in bond performance with respect to steel is accounted for by introducing a bond reduction factor (C_b).

Table 1 summarizes design parameters for CFRP-PC according to a selection of international guidelines. The values used for the HRB design align with the FDOT Structures Manual (FDOT, 2018a), Construction Specifications (FDOT, 2018b) and Structures Index (FDOT, 2016a; FDOT, 2016b). Table 2 summarizes design parameters for GFRP-RC according to a selection of international guidelines. The values used for HRB design align with the proposed AASHO BDGS-GFRP (AASHTO, 2018).

In the following, High-Strength Stainless Steel HSSS will be considered for comparison with CFRP in pre-stress applications. In the United States, the material is not regulated at the federal level, but is included in the FDOT Structures Manual (FDOT, 2018a), Construction Specifications (FDOT, 2018b), and Structures Index (FDOT, 2016a; FDOT, 2016b).

Sea-concrete is not regulated at either the State or Federal level. FDOT Construction Specifications (FDOT, 2018b) were used where applicable and modified accordingly. RCA and RAP nonstructural concrete mixes are regulated by FDOT Construction Specifications (FDOT, 2018b).

Table 1. Design parameters for CFRP-PC.

	JCSE	CNR	*fib* MC	ACI 440.4R	FDOT	AASHTO CFRP
	1997	2007	2013	2004	2018	2018$^{(1)}$
ϕ_t	0.87	0.60	0.80	0.85	0.85$^{(4)}$	0.75
C_E	1.00	0.90	0.90$^{(2)}$	0.90 $^{(3)}$	0.90$^{(3)}$	1.00
C_J	0.80	-	-	0.65	0.70	0.75
C_C	0.70	0.90	0.80	0.60	0.60$^{(4)}$	0.70
C_b	-	0.59	0.71$^{(2)}$	-	0.71$^{(3)}$	-

Table 2. Design parameters for GFRP-RC.

	JCSE	CNR	*fib* MC	ACI 440.1R	FDOT	AASHTO GFRP
	1997	2007	2013	2015	2018	2018$^{(1)}$
ϕ_t	0.77	0.60	0.80	0.55	0.55$^{(3)}$	0.55
C_E	1.00	0.70	0.55$^{(2)}$	0.70	0.70$^{(3)}$	0.70
C_f	-	-	-	0.20	0.20$^{(3)}$	0.25
C_C	0.50$^{(5)}$	0.30	0.30	0.20	0.20$^{(3)}$	0.30
C_b	0.77	0.59	0.71$^{(2)}$	0.71	0.71$^{(3)}$	0.83

(1) under consideration.
(2) from *fib* bulletin 40 (*fib,* 2007).
(3) from ACI 440.1R (ACI, 2015).
(4) from ACI 440.4R (ACI, 2004).
(5) from JSCE-CES23 commentary (JSCE, 1997)

3.2 *Material Properties*

FRP is a brittle composite material, elastic until failure, stronger, but less stiff with respect to Mild Carbon Steel (MCS) and High-Strength Carbon Steel (HSCS). The guaranteed strength of an FRP bar or strand is defined as the experimental average value minus three standard deviation (ACI, 2015). The approach is more conservative with respect to the calculation of characteristic strengths for steel reinforcement and concrete, defined as the average minus 1.64 standard deviations under the assumption of normal distribution (CEN, 2008). The minimum specified values for strength and stiffness of GFRP bars are defined by ASTM D7957 (ASTM, 2017a). CFRP strands and HSSS strands are not regulated at the federal level, but the FDOT Construction Specifications (FDOT, 2018b) include minimum specified values for strength and references to applicable acceptance criteria. HSCS strands are regulated by ASTM A416 (ASTM, 2017b), whereas MCS bars are regulated by ASTM A615 (ASTM, 2016).

Tables 2 and 3 report the experimental and specified properties for the CFRP strands (SML, 2017a) and M13 GFRP bars (SML, 2017b) deployed in the HRB. Table 4 reports the specified properties of certified HSSS strands (FDOT, 2018b). Figure 3 compares the mechanical properties of CFRP, GFRP, HSSS, HSCS, and MCS. HSSS performance is closer to CFRP rather than HSCS. This includes a quasi-brittle behavior with no plastic plateau.

Experimental properties for FRP products lay above the minimum specified values. The general conservativeness in approaching FRP characterization and design is derived by the historically limited confidence in deploying non-ductile innovative materials with respect to traditional solutions. Rationalization of design requirements is a major area of advancement for FRP-RC/PC. The new generation of design regulations is moving toward relaxing historic overconservative assumptions (Rossini et al., 2018a).

All the prestressed elements of the bridge entail equivalent C40/50 concrete. Bulkhead cap sea-concrete mix is designed for a compressive strength of 38 MPa at 28 days. RCA and RAP concrete are equivalent C16/20 nonstructural mixes.

Table 3. Mechanical properties for 15.2 mm CFRP strands.

CFRP 15.2 mm	Stress /	MPa	ε_f %	F_f kN	E_f GPa	a_f mm^2
Mean	f_{fm}	3267	2.08	378	157	115
Guaranteed	f_{fu}^*	2990	1.90	346		
Specified	f_{fu}'	2336	1.51	270	155	116
Design	f_{fd}	2102	1.36	243		
Jacking	$f_{f,j}$	1471	0.95	170		
Sustained	$f_{f,s}$	1261	0.81	146		

Table 4. Mechanical properties for M13 GFRP bars.

GFRP M13	Stress /	MPa	ε_f %	F_f kN	E_f GPa	a_f mm^2
Mean	f_{fm}	983	1.57	125	62.7	147
Guaranteed	f_{fu}^*	883	1.41	112		
Specified	f_{fu}'	758	1.69	96	44.8	127
Design	f_{fd}	530	1.18	67		
Jacking	$f_{f,j}$	n.a.	n.a.	n.a.		
Sustained	$f_{f,s}$	160	0.36	20		

Table 5. Mechanical properties for 12.7 mm HSSS strands.

HSSS 12.7 mm	Stress /	MPa	ε_s %	F_s kN	E_s GPa	a_s mm^2
Ultimate	f_{su}	1724	1.10	170	162	99
Yielding	f_{sy}	1606	1.00	159		
Jacking	f_{sj}	1293	0.80	128		

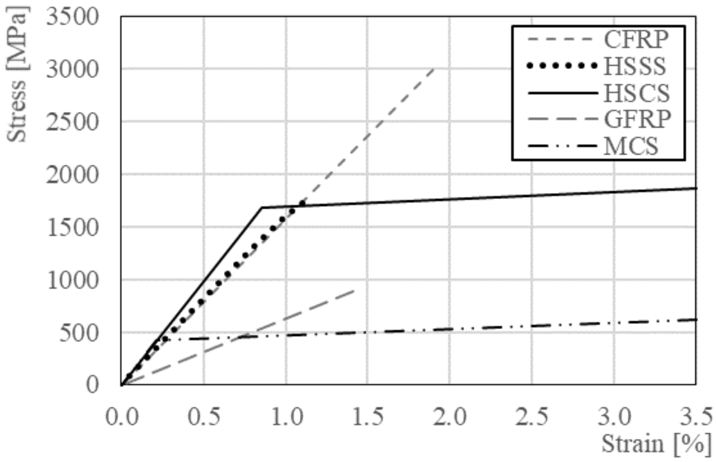

Figure 2. Mechanical properties of various reinforcement materials.

4 STRUCTURAL MEMBERS

4.1 *CFRP-PC bearing piles*

The Halls River Bridge comprises a total of 36 CFRP-PC bearing piles, divided over 6 bent caps. The piles are designed according to FDOT Index series 22600 (FDOT, 2016a) for a compressive strength of 3254 kN and a flexural strength of 325 kN-m. The cross section is squared with a side of 460 mm (Figure 3a). Prestress is applied through 12 CFRP strands of 15.2 mm diameter tensioned at 151 kN each. Confinement is provided by a spiral of 5 mm CFRP single wire with a pitch varying from 25 mm to 76 mm to 152 mm as showed in Figure 3b. Whereas next generation design guidelines suggest concrete clear cover values of 20 mm to 38 mm, the FDOT Structures Manual (FDOT, 2018a) imposed a concrete clear cover of 76 mm for compatibility with traditional prestressing patterns. The value coincides with standard practice for steel reinforcement in aggressive subtropical environment.

Table 6 compares three different reinforcement alternatives for the same pile configuration. The cost column reports the strand material cost per linear meter of pile, as estimated by FDOT (2015). P_i is the initial prestress on each strand. ΔP includes the total prestress losses as a percentage of the initial prestress. σ_∞ is the level of stress in the concrete after losses at t_∞. Prestress efficiency (e_p) is computed as the ratio of retained prestress at t_∞ over the initial prestress. CFRP stands up as the most efficient alternative and is less expensive with respect to HSSS. The higher material cost with respect to HSCS strands is expected to be absorbed over the long term thanks to the superior durability of CFRP (Cadenazzi et al., 2018a).

The piles were designed to be driven by impacting hammer to a depth of 19 m, for a total length of 21 m. Unexpected geotechnical conditions forced driving the three piles of one of the intermediate bents to a depth of 46 m requiring splicing of additional segments (Cadenazzi et al., 2018b). MSS M25 bars were used as dowels, whereas CFRP dowels are undergoing validation to be ready for the second construction phase. Figure 4 shows pile driving and splicing.

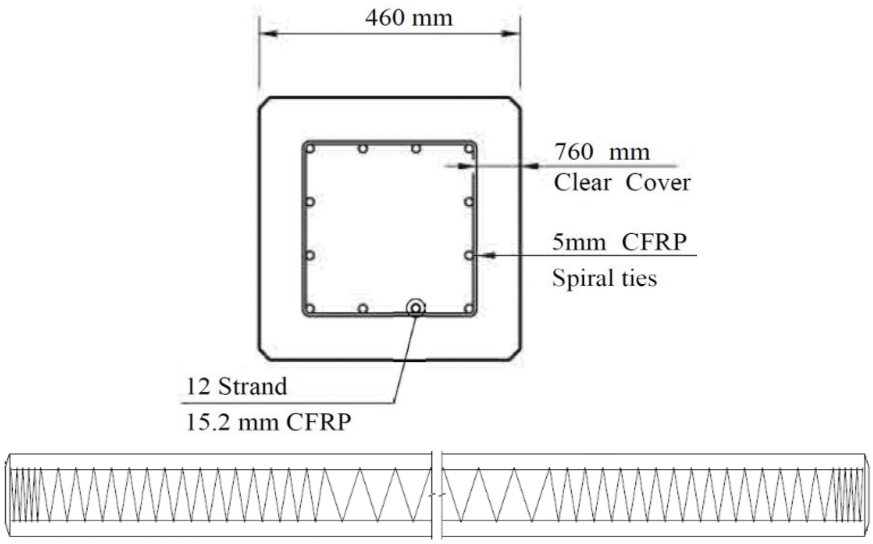

Figure 3. Bearing pile cross section (a) and lateral view (b).

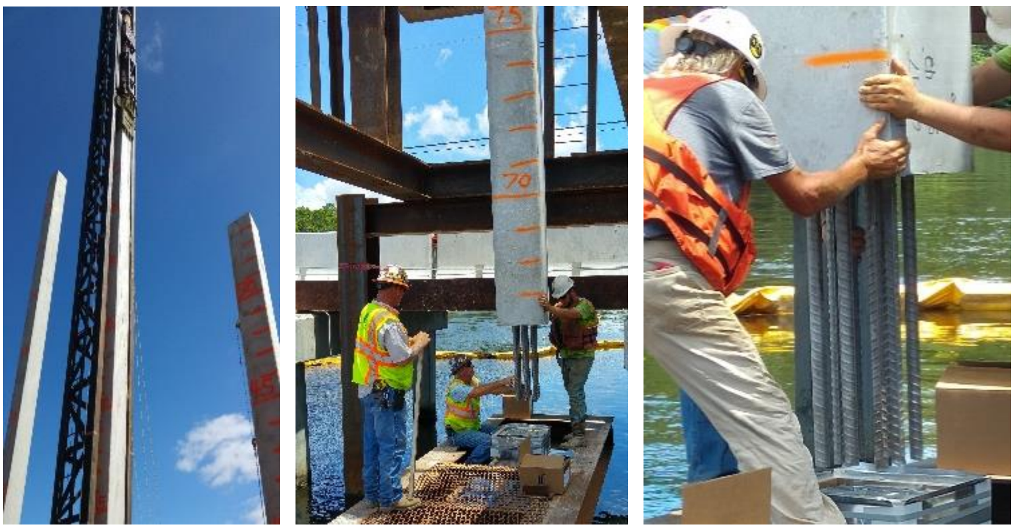

Figure 4. Bearing pile driving and splicing operations.

Table 6. Comparison of different reinforcing material for the same sheet pile configuration.

	Strands	Cost	P_i	ΔP	σ_∞	e_p
	n°	\$/m	kN	%	MPa	-
CFRP	**12**	**150**	**151**	**16%**	**7.3**	**0.84**
HSSS	16	210	116	18%	7.2	0.82
HSCS	12	40	156	24%	6.8	0.76

4.2 *CFRP-PC/GFRP-RC sheet piles*

The Halls River Bridge comprises a total of 149 CFRP-PC/GFRP-RC sheet piles located in direct contact with saltwater. Furthermore, a total of 86 Hybrid HSCS-PC/GFRP-RC sheet piles are located in the less exposed portion of the retaining walls. The piles are designed according to FDOT Index 22440 (FDOT, 2016b) for a compressive strength of 3904 kN and a flexural strength of 224 kN-m. The cross section is rectangular with a width of 760 mm and a depth of 300 mm. The wall system is modular with male-female connections (Figure 5a). Prestress is applied through 8 CFRP strands of 15.2 mm diameter tensioned at 170 kN each. Confinement is provided by M13 GFRP ties with a pith varying from 102 mm to 305 mm as showed in Figure 5b. Concrete clear cover is set to 76 mm.

Table 7 compares three different reinforcement alternatives for the same sheet pile configuration. As discussed for the case of bearing piles, CFRP stands up as the most efficient alternative and is less expensive than HSSS. The higher cost with respect to HSCS is expected to be absorbed thanks to the superior durability of CFRP (Cadenazzi et al., 2018a).

The sheet piles were designed for installation by water-jetting to a depth of 8 m for a total length of 9 m in a cantilever configuration. The unexpected presence of a layer of hard limestone reduced the installation depth to approximately 4 m for relevant portions of the wall. To guarantee the required strength and stiffness to the retaining wall, an anchored sheet pile variant was adopted. The sheet piles were cut to length and the cut-off portions were installed as deadmen. The sheet pile bulk head cap was connected to the deadmen through MSS bars mildly tensioned using screw couplers (Cadenazzi, 2017). Figure 6 shows the installation of the sheet piles.

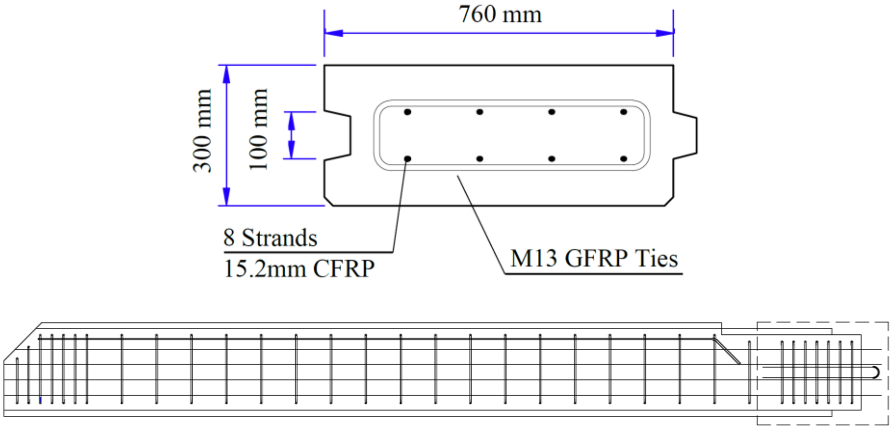

Figure 5. Sheet pile cross section (a) and lateral view (b).

Figure 6. Sheet piles during installation

Table 7. Comparison of different reinforcing material for the same sheet pile configuration.

	Strands	Cost	P_i	ΔP	σ_∞	e_p
	n°	$/m	kN	%	MPa	-
CFRP	**8**	**100**	**170**	**11%**	**5.2**	**0.89**
HSSS	14	183	116	16%	5.8	0.84
HSCS	10	33	245	16%	8.8	0.84

4.3 *GFRP-RC pile bent caps.*

The Halls River Bridge comprises 6 GFRP-RC cast-in-place pile bent caps of rectangular cross section having a width of 1.22 m and a depth of 0.91 m (Figure 7). Each bent cap is under-reinforced with failure controlled by FRP rupture. Design is governed by service considerations, as typical for the case of GFRP-RC. Since the provisions for this type of element are particularly conservative, new provisions have been proposed for the next generation of design guidelines.

The design of the bent cap of the HRB according to ACI 440.1R (ACI, 2015) and the first edition of the AASHTO BDGS-GFRP (AASHTO, 2009) is discussed by Rossini et al. (2018a) and requires 16 M25 GFRP bars to resist a factored moment of 575 kN-m. This corresponds to an area of reinforcement 2.7 times the area of steel reinforcement required for an equivalent member. According to the provisions of the second edition of the AASHTO BDGS-GFRP (AASHTO, 2018), the bent cap design requires 9 M25 GFRP bars. This corresponds to a reduction of 40% with respect to the first edition.

Figure 8 shows the number of M25 bars required for the bent cap according to the first and second editions of the AASHTO BDGS-GFRP. The quantities are plotted in parametric diagrams as a function of the bond reduction factor ($C_b = 1/k_b$). Further details on the approach adopted can be found in Rossini et al. (2018a). The two diagrams show how a rationalization of the sustained and cyclic demand, along with a slight relaxation of design factors per Table 2, result in a decrease of the amount of reinforcement required. According to the second edition of the AASHTO BDGS-GFRP, crack width governs over cyclic fatigue, creep rupture, and strength. Further savings are expected to come from improved GFRP bond performance, as expressed by the bond reduction factor ($C_b = 1/k_b$) (see Table 2).

Table 8 summarizes the various design alternatives for the bent cap, including the selected option and a comparison to an equivalent MCS-RC member. The higher material cost is expected to be absorbed over the long period thanks to the superior durability of GFRP (Cadenazzi et al., 2018a). Challenges and opportunities related to the use of GFRP in the construction of the bent caps are discussed by (Cadenazzi et al., 2018b).

4.4 *GFRP-RC girders*

The Halls River Bridge comprises 45 Hillman Composite Beams (HCB) consisting of a composite GFRP shell encasing a steel-reinforced concrete shallow tied-arch and lightweight filling foam (Figure 9). This complex structural solution is reinforced with 18 galvanized unstressed steel strands of 12.7 mm diameter and connected to the deck through galvanized steel connectors. The composite shell is intended to enhance durability of the RC structural core (Aboelseoud & Myers, 2016). Corrosion of the galvanized steel connectors in a more aggressive environment is an unaddressed concern.

An alternative GFRP-RC precast solution is discussed in this section, designed to ensure the same level of strength and performance in addition to validated durability performance (ASTM, 2017a). Maintaining the original rectangular section of the HCB with a width of 590 mm and a depth of 530 mm, a total of 18 M25 GFRP bars is sufficient to resist against a factored moment of 1085 kN-m. The girder is connected to the deck through M13 GFRP shear connectors in the shape of closed stirrups spaced 50 to 200 mm. The member complies with the AASHTO BDGS-GFRP 2nd edition (AASHTO, 2018), whereas design according to the first edition of the guideline is not permitted. Figure 10 visualizes the level of strains in the member under different loading conditions. Given the conservativeness of the strength reduction factor for FRP-controlled failures (ϕ_t), the strains under Ultimate Load (Ultimate L.) lay below the flexural failure threshold defined as Nominal Resistance (Nominal R.) by a margin of about 50%.

The GFRP-RC precast solution discussed has a total cost estimated at $430 per linear meter per member, corresponding to a portion of the cost of HCB estimated at $1580 per linear meter per member (Cadenazzi et al., 2018a). The challenges and opportunities related to HCB installation are discussed by Cadenazzi et al. (2018b).

Figure 7. Pile bent cap after demonlding.

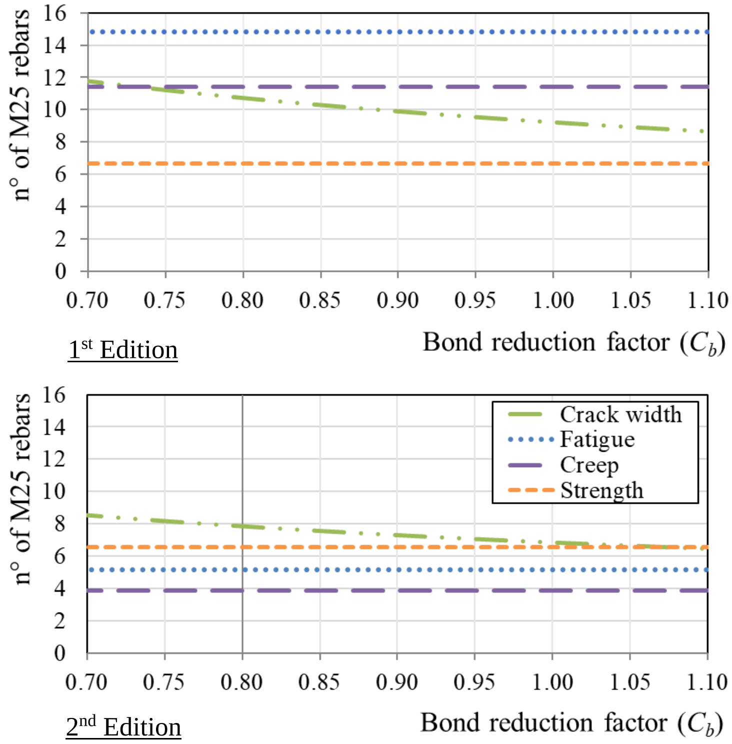

Figure 8. Required amount of reinforcement for the pile bent cap according to AASHTO BDGS-GFRP.

Figure 10. HCB during installation.

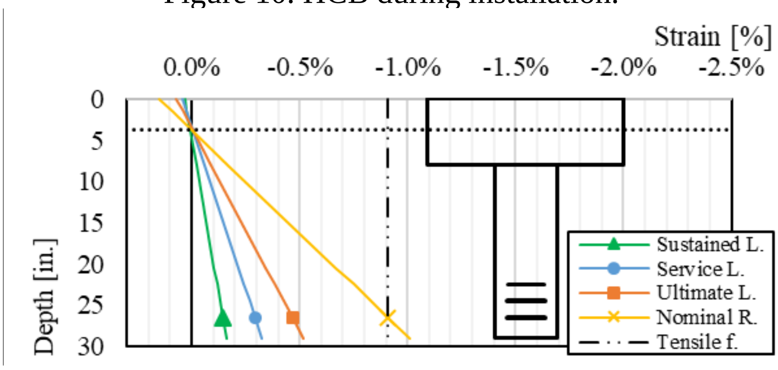

Figure 10. GFRP-RC girder sectional analysis. Geometry not to scale.

Table 8. Comparison of different reinforcement layout for the same bent cap configuration.

	M25 Bars n°	Cost $/m	GFRP to steel ra-
BDGS-GFRP 1st	16	61	2.7
HRB Design	**12**	**46**	**2.0**
BDGS-GFRP 2nd	9	34	1.5
Mild Carbon Steel	6	20	1.0

4.5 *GFRP-RC deck, slabs, railings and bulkhead caps.*

The entire structure of the bridge, including secondary element and the entire superstructure, is reinforced with corrosion-resistant and hybrid solutions. The GFRP-RC deck is designed according to the first edition of AASHTO BDGS-GFRP (AASHTO, 2009). Decks tend to undergo less corrosion in Florida, not being exposed to deicing agents nor being in contact with seawater. The choice of GFRP for the deck was made for consistency and concept demonstration.

The GFRP-RC railings were designed according to FDOT Index D22420 (FDOT, 2017a). Railings design is discussed by Rocchetti et al. (2018).

The GFRP-RC bulkhead cap was design according to FDOT Index 22440 (FDOT, 2016b). The GFRP-RC approach slabs were designed according to FDOT Index D22900 (FDOT, 2017b). In these cases, corrosion is a major problem given the exposure to saltwater in the tidal zone and soil, respectively.

The 2nd edition of the AASHTO BDGS-GFRP (AASHTO, 2018) covers the design of every component of a GFRP-RC bridge, including substructure and continuous railings. The provisions align with FDOT Structures Manual (FDOT, 2018a), Construction Specifications (FDOT, 2018b), and Structures Index (FDOT 2016a, 2016b, 2017a, 2017b).

5 SUMMARY AND CONCLUSIONS

In this study, HRB is presented in its role of demonstrator for a series of innovative material and structural solutions part of the SEACON research project and related efforts. Furthermore, HRB served as a proof-of-concept for the development of a new generation of design and construction guidelines for FRP-RC/PC structures.

1. The SEACON research project and its outcomes in terms of development of innovative concrete mixes and validation and deployment of non-corrosive reinforcement have been discussed.
2. The design of CFRP-PC bearing piles, CFRP-PC/GFRP-RC sheet piles, GFRP-RC pile bent caps, and GFRP-RC girders has been presented.
3. CFRP strands proved their superior structural efficiency and appealing economic implications with respect to corrosive (HSCS) and non-corrosive (HSSS) metallic alternatives.
4. The appealing economic implications and validated durability performance of GFRP-RC have been discussed. Costs are in the order of 1/3 with respect to alternative corrosion-resistant solutions (e.g. HCB).
5. The implications of the second edition of the AASHTO BDGS-GFRP on GFRP-RC design have been discussed. This includes both reinforcement savings in the order of 40% with respect to the first edition (e.g. GFRP-RC bent cap) and the possibility to use GFRP reinforcement for new members (e.g. GFRP-RC girders).
6. A comprehensive state-of-the-practice review for FRP design and application in bridges has been provided.

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A Seawall Constructed with GFRP Bars as Structural Reinforcing

The system was designed to help preserve the character of the local environment and reduce life-cycle costs

by Christian C. Steputat, Steven Nolan, Lowry Denty, Paul A. Kaminski, and Antonio Nanni

In 2016, the seaside community of Flagler Beach, FL, was one of the many communities severely impacted and battered by Hurricane Matthew. To help mitigate the effects of future storms, a secant-pile seawall is being constructed east of State Road A1A (SR A1A) at Flagler/Beverly Beach in the northeastern section of Flagler County, FL. When completed, the new barrier will extend 1.5 km (almost 5000 ft).

To ensure that the protective system remains resilient for decades, the project is being built using concrete reinforced with glass-fiber-reinforced polymer (GFRP) bars. The secant piles have been completed, and the seawall pile cap with vegetative cover is expected to be substantially completed by October 2019.

Background

SR A1A is especially important because it has been designated an evacuation route. Unfortunately, it also passes through areas that are highly vulnerable to hurricane surge flooding. As a result of extensive corrosion of an existing steel sheet pile bulkhead and erosion of an adjacent dune system, almost 1 mile (1.6 km) of the highway collapsed during Hurricane Matthew.

To ensure the restored highway remains in service in the event of a similar future storm, a protection and support system is under construction in the stretch indicated as Segment 3 in Fig. 1. The project consists of a GFRP-reinforced concrete secant-pile seawall/bulkhead designed to provide support for the highway in the event the adjacent sand dunes are lost during a major storm. Figure 2 illustrates the seawall with a reestablished dune that helps to preserve the character of the local environment, including the "Old Florida" feeling of Flagler Beach.

Specifications and Standards

The engineering design specifications and construction standards for this seawall/bulkhead with dune reestablishment comprised information from several documents, including:

- Florida Department of Transportation (FDOT) Structures Manual, January 2018,[1] and subsequent Standards Design Bulletins;
- FDOT Design Manual, January 2018[2];
- American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor (LRFD) Bridge Design Specifications, 8th Edition,[3] and all subsequent interims;
- FDOT Standard Specifications for Road and Bridge Construction, July 2018,[4] Divisions II and III; and
- ACI 440.1R-15, "Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars."[5]

Per FDOT Specifications, Section 346,[4] concrete was to be Class IV without silica fume, and the 28-day compressive strength for the cast-in-place secant-pile seawall cap was specified as 5500 psi (38 MPa). The efficiency and production performance rates of secant-pile installations are highly dependent on the availability and performance of the pile grout.

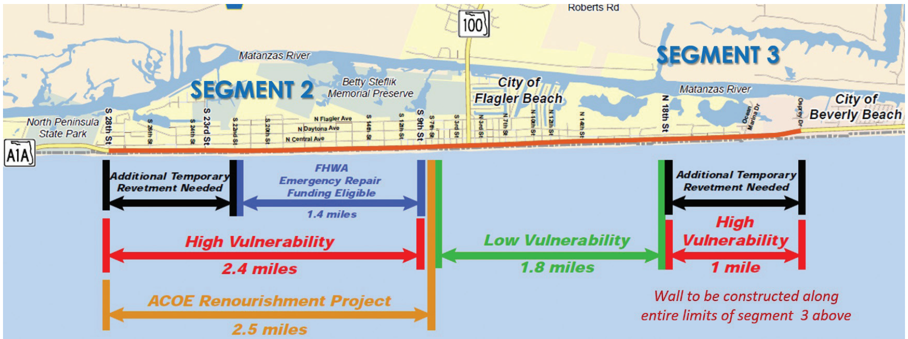

Fig. 1: The secant-pile seawall/bulkhead project described in this article is located within Segment 3, an area that is considered highly vulnerable to hurricane surge flooding

Per FDOT Specifications, Section 455, Index E,[4] grout needs a minimum standard flow rate of 15 seconds and achieve a minimum compressive strength of 4000 psi (28 MPa). In addition, a cased, continuous-flight auger or equivalent was specified; all pile centers were to be located to an accuracy of 1-1/2 in. (38 mm) in plan; and piles were to be installed using a concrete guide wall that was to be removed after pile installation and prior to cap installation.

The project was designed for an extremely aggressive marine environment, with chlorides at 1320 ppm. This led to a specified minimum resistivity of 300 Ω·cm. The cast-in-place pile cap and auger-cast piles were designed for a minimum cover of 3.0 in. (75 mm), while placement tolerances were per FDOT Specifications, Section 415.[4] The GFRP bars used in this project were required to meet FDOT Specifications, Section 932-3, Table 3-4.[4]

The seawall was designed using strength design with live loads of 220 psf (10.5 kPa) and dead loads of 150 lb/ft^3 (2400 kg/m^3).

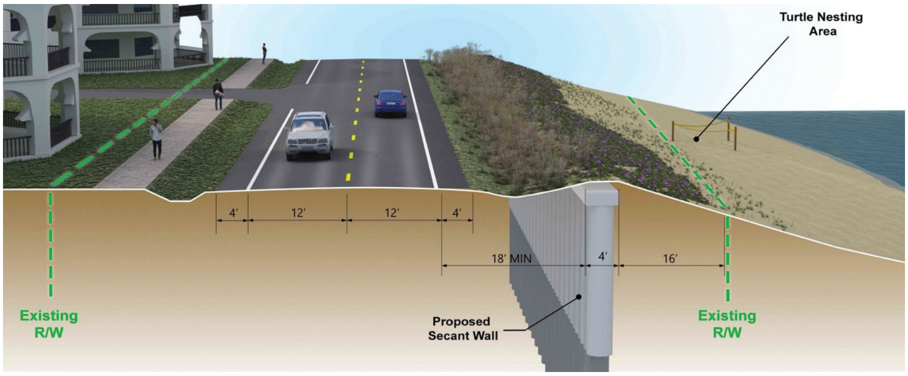

Fig. 2: A schematic rendering of the SR A1A secant-pile seawall/bulkhead project with dune reestablishment and existing right-of-way (r/w) limits

GFRP Laboratory Testing

The evaluation of GFRP reinforcing bars was conducted at the Civil, Architectural, and Environmental Engineering Advanced Structures and Materials Laboratory (SML) at the University of Miami, Coral Gables, FL. The laboratory maintains a quality assurance and quality control system (QA/QC) in compliance with the requirements of ISO 17025-2017,[6] accredited under the International Accreditation Services (IAS), and is a qualified testing laboratory for FDOT projects. Table 1 shows test results for the GFRP No. 8 bars tested in 2019 on February 26 (Lot 1), April 16 (Lot 2), and March 18 (Lot 3).

GFRP Cages and Pile Caps

The seawall's auger-cast concrete secant piles are 36 in. (910 mm) in diameter. Primary and secondary piles are 36 and 20 ft (11 and 6 m) in length, respectively. Primary piles are reinforced with 25 No. 8 GFRP bars; secondary piles are reinforced with only a single, centrally placed No. 8 GFRP bar. The design called for a pile overlap of 4 in. (102 mm) and a 4 ft (1.2 m) wide, 18 in. (457 mm) deep pile cap over the full length of the secant-pile seawall (1.5 km). GFRP reinforcing cages for the primary piles are shown in Fig. 3.

GFRP constructability

The project's specialized drill rig was assembled on Flagler Beach starting on

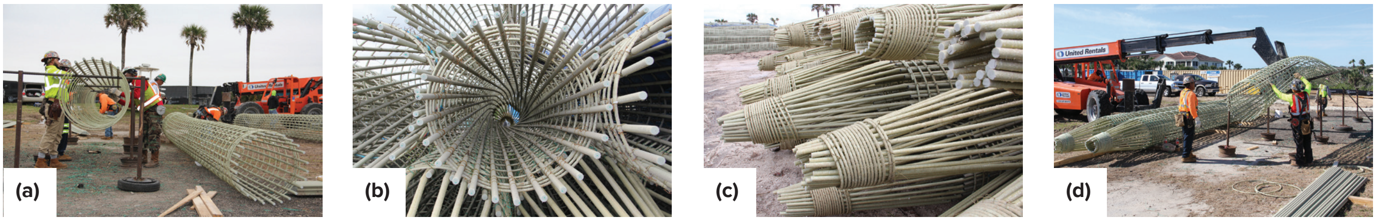

Fig. 3: The seawall was constructed using GFRP cages: (a) cage assembly was completed in the field; (b) a conventional bar layout was used in the upper sections of a cage; (c) a special pile-toe assembly was used in the lower section of each cage; and (d) assembled cages were moved from the assembly stands using a telescopic forklift

Table 1:
GFRP bars were shown to exceed the FDOT Specifications[4]

Test prefix	Standard test method	Laboratory test descriptions	Laboratory test results: Lot 1	Lot 2	Lot 3	Mean	FDOT Section 932-3, Table 3-4 requirements[4]
DSC	ASTM E2160[7]	Degree of cure, %	98	100	100	99.3	≥ 95
	ASTM D3418[8]	Glass transition temperature, °F	225	275	256	252	≥ 212
FC	ASTM D2584[9]	Fiber content (by weight), g	84	84	84	84	≥ 70
MAS	ASTM D570[10]	Moisture absorption (short term), %	0.17	0.18	0.12	0.16	≤ 0.25
MXA	ASTM D792[11]	Measured cross-sectional area for No. 8 bar, in.^{2}	0.818	0.825	0.803	0.815	> 0.738 < 0.913
TNS	ASTM D7205/D7205M[12]	Guaranteed tensile load, kip	111.4	93.6	104.3	103.1	> 66.8
		Tensile modulus of elasticity, ksi	8280	7980	7600	7950	≥ 6500

Notes: °C = (°F − 32) / 1.8; 1 g = 0.04 oz.; 1 in.^{2} = 645 mm^2; 1 kip = 4.4 kN; 1 ksi = 6.9 MPa

February 25, 2019. Over the following 4.5-month period, 1847 piles were installed.

The secant piles were located using a guide wall—a plain concrete template cast in a trench. Pile locations were set in the template using removeable, preassembled steel formwork (Fig. 4). In addition to helping define the verticality and top elevation of the secant piles, the template served as a removable form that exposed the upper reaches of the piles and allowed them to extend into the subsequently placed seawall/bulkhead pile cap.

Secant-pile installation

The secant-pile construction sequence involved drilling and placing secondary piles (piles reinforced with only a single GFRP bar) on Day 1 and Day 2 of a 3-day rotating cycle. On Day 3, primary piles (those reinforced with 25 No. 8 GFRP bars, spiral hoops, toe-assembly, and ties) were drilled and placed. This placement sequence resulted in an interlocked and relatively homogeneous deep foundation wall-type system (shown in Fig. 5). Additionally, due to the fact that primary piles (piles with the full reinforcing GFRP bar cages) are cut into the secondary piles (those with only a single, central GFRP bar), all the reinforcing GFRP cages are provided with full cover.

GFRP cage installation

The lightweight, flexible GFRP reinforcing cages in the primary piles allowed for smooth and rapid installation (Fig. 6). Due to the relatively loose nature of the Flagler Beach "beach sand," the piles were installed using the cased auger-cast method. The secant-pile and pile-cap installations were also closely monitored and timed. Based on current estimates, in comparison with similar projects comprising steel reinforcing bar cages, the GFRP cages resulted in bar placement time savings of 32 to 52% throughout the typical observation time of an extended workweek. More time savings were noted toward the end of each week.

Figure 7 shows the installed secant piles, continuous pile-cap construction, and dune reestablishment over the constructed secant-pile seawall. The reestablished dune will minimize the environmental and aesthetic impacts of the protective structure.

Summary and Discussion

The SR A1A seawall project comprises GFRP reinforcing bars in its 1847 secant piles and continuous cap. The GFRP bars have a high tensile strength (about twice the strength of steel bars), low weight (about one quarter the weight of steel bars), and are noncorrosive. As a result of these features, the cage installations were smooth and rapid, and maintenance and repair costs over the life cycle of the seawall are expected to be minimal. While periodic restoration of the dune may be needed to minimize the potential for scouring of the seawall, the durable materials in the wall will provide an extended time window for restoration activities.

Fig. 4: Construction of the guide wall, a concrete template that was used to align and locate the seawall's secant piles and provide positional restraint and the required verticality of the piles. The guide wall was removed prior to construction of the pile cap

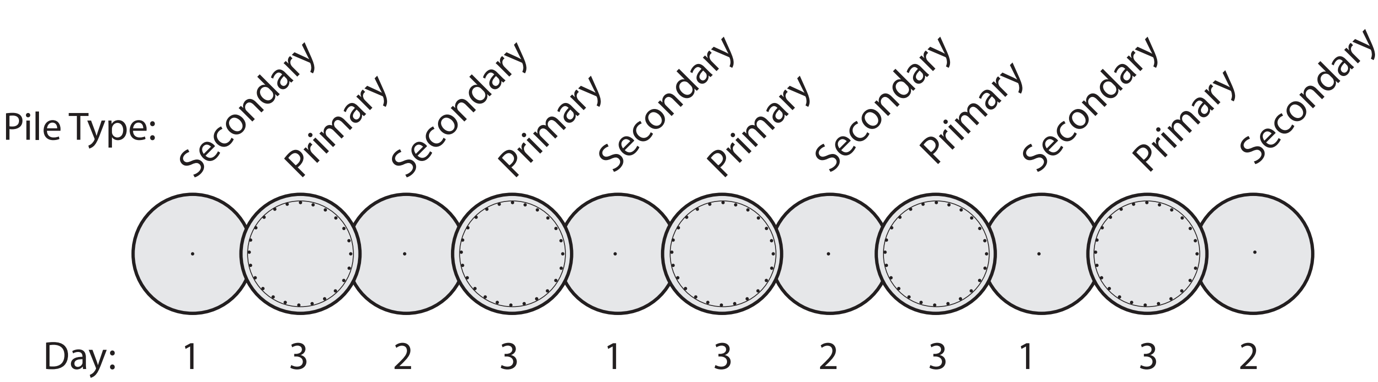

Fig. 5: The piles were placed in a sequence that ensured that the primary piles could be drilled into and interlocked with the secondary piles

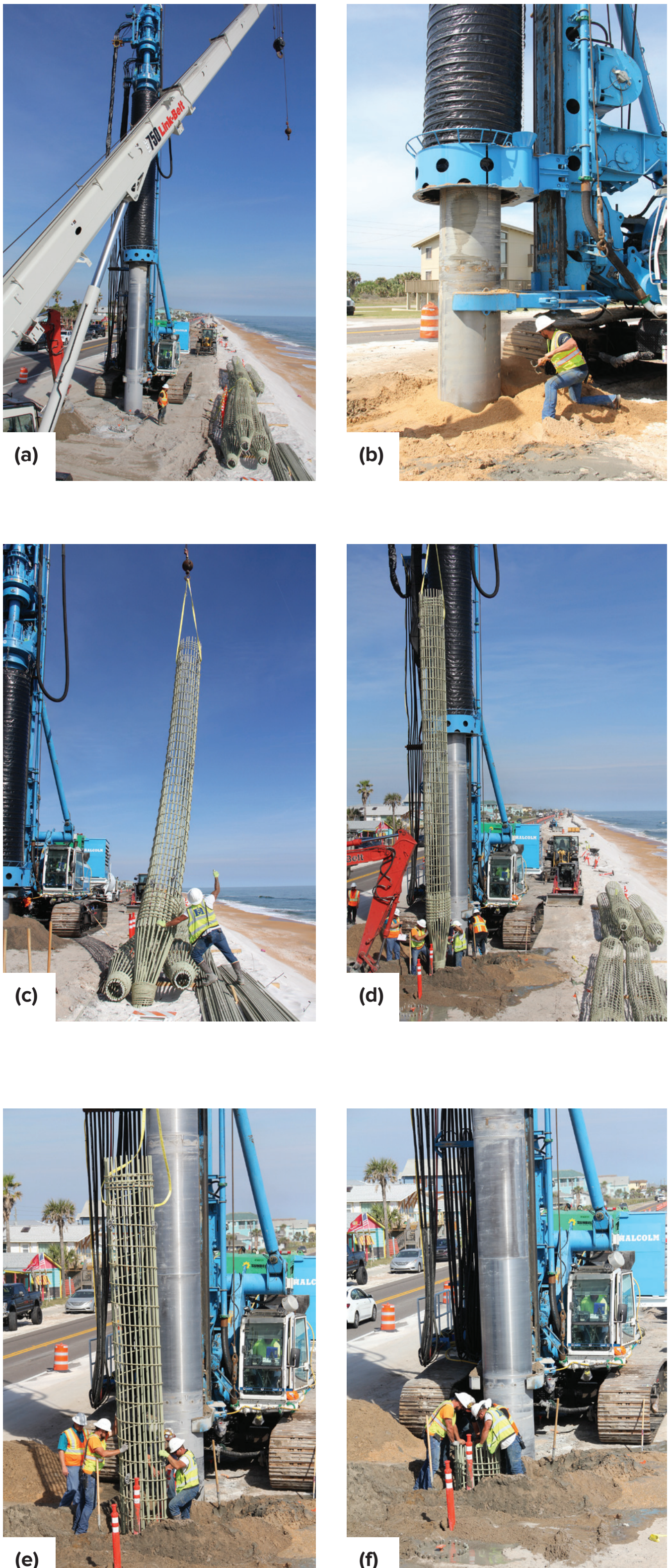

Fig. 6: A specialized drill rig was used to install the secant piles using the cased auger-cast method. The installation sequence comprised: (a) drilling; (b) soil removal and grouting; (c) GFRP cage lifting; (d) cage alignment; (e) cage insertion into the grout; and (f) final cage positioning

As the interest in and use of GFRP reinforcement for concrete structures increases further, all stakeholders are working on different fronts to make the technology more effective and efficient, while maintaining low cost and durability as the essential attributes.

Acknowledgments

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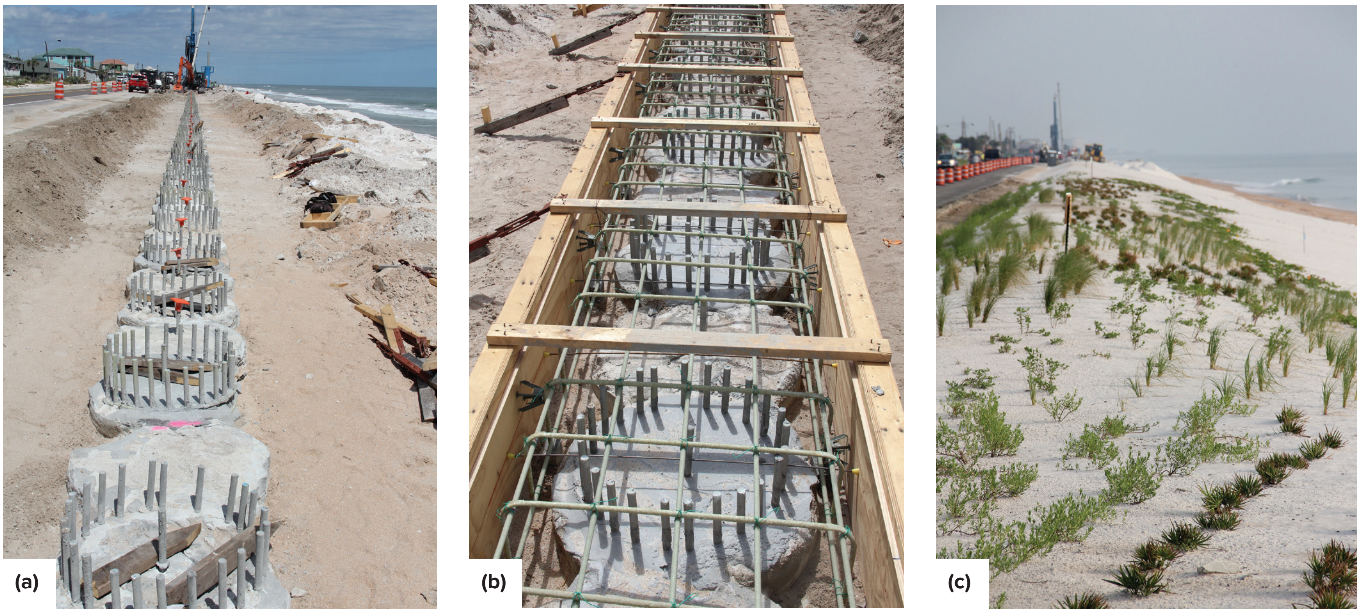

Fig. 7: After secant piles were installed: (a) the concrete guide wall was removed; (b) formwork and GFRP reinforcing bars were placed for the pile cap; and (c) a dune was constructed over the wall after the pile-cap concrete was placed and cured, and the pile-cap formwork was removed

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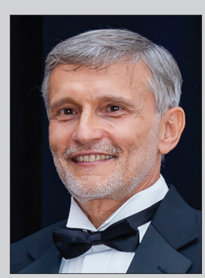

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177 *Seawall-Bulkheads, SEACON, Sustainability and Resilience*

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Keywords: Bulkhead, Composites, FRP Reinforcing, Prestressing, Seawall, Sheet pile wall.

Abstract

Florida was an early adopter of concrete seawalls and bulkheads in sheltered marine environments. Land booms in the 1920's, 1950's, and 1960's lead to extensive development of the coastal and estuarine waterfront. Government environmental regulations in the 1970's slowed much of the shoreline armouring, but not before an estimated 6,000 km of reinforced and prestressed concrete seawalls “protected” the urban coastline of Florida. The legacy of these events in combination with an aggressive subtropical environment is an extensive maintenance, rehabilitation or replacement (MRR) liability, and many of these structural systems are beyond their intended design-life. The major share of the MRR liability will be borne by private owners (estimated at 80%). The limited understanding or expectation of the durability limits for these legacy systems at the time of installation necessitates considering a different approach for investment of future resources. FDOT collaborated with the SEACON project in the development and application of advancements in materials technology for effective deployed from a durability, economic and sustainability perspective. Primarily focused on fibre-reinforced polymer (FRP) and stainless-steel reinforcing, in combination with seawater concrete and chloride contaminated aggregate, the SEACON research team is a partnership between US and Italian academic, commercial and government transportation stake-holders. Field demonstrations of real world structures utilizing these components have been constructed, documented and are being monitored. FDOT developed standards for carbon-FRP and high-strength stainless-steel, and is considering an emerging glass-FRP prestressing technology, for broader deployment of bridge piling, seawall construction and rehabilitation. This presentation presents a culmination of these efforts and a vision applicable to many coastal communities worldwide.

1 Introduction

Florida was an early adopter of precast concrete sheet piles for seawall-bulkheads in sheltered marine environments. The sunshine state land boom of the 1920's lead to extensive development of coastal and estuarine waterfront as property speculators attempted to maximize boating access, water views and escalate property values. During this period, precast reinforced concrete (RC) sheet piles and panels where used in combination with dredge-and-fill operations to create valuable waterfront properties (Figure 1a). The Great Depression and World War II halted much of this development, but resurgence with the 1950's housing boom created a market for the newly adopted technology of prestressed concrete (PC) with the promise of greater durability and strength. The development of finger islands was also a common practice used by developers in this period to maximized the amount of water access with "canal lots" (Figures 1b & 1c). Coastal development and shoreline armoring continued in Florida through the 1960's spurred along no doubt by the availability of air conditioning (Badger & Blinder 2017) and a burgeoning consumer class.

It was not until the 1970's with the adoption of the 1972 US Coastal Zone Management Act (US Congress 1972a) and the rewrite of the US Clean Water Act (US Congress 1972b), that greater restrictions than just economy were place on coastal land developers. Presently there is estimated by the authors to be approximately 3,600 miles (5,800 km) of RC and PC seawall-bulkheads armoring the coastline of Florida. Accurate estimates are difficult to obtain due to the highly fragmented and localized regulatory oversight. The majority of seawall-bulkheads protect private property, estimated at 80%, with the remainder split between, municipal, state and federal government authority owners.

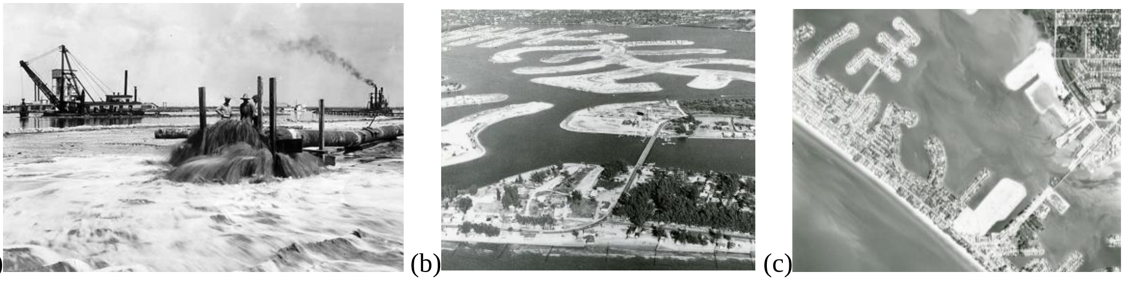


Fig. 1 Examples of land reclamation in Florida between 1920 and 1970: (a) Davis Island-Tampa Bay, 1924, dredge-and-fill operation [1]; (b) Pinellas county, 1950's – Boca Ciega Bay finger island construction [2]; (c) Pinellas county, 1950's – Treasure Island Causeway [2].
[1] Photograph courtesy of the Burgert Bros.
[2] Photographs courtesy of Pinellas County, Heritage Village, Archives and Library.

2 The land boom shoreline legacy

The legacy of extensive shoreline development, expansion and hardening, in combination with Florida's extremely aggressive subtropical environment is a tremendous maintenance, rehabilitation and replacement (MRR) liability. Many of these seawall-bulkheads have aged beyond their original design life and coupled with the increasing threat of sea level rise (SLR) the remaining structures may need significant rehabilitation or replacement in the near future. The benefits of maintaining or improving these legacy structures was recently identified in a report by the Union of Concerned Scientist (UCS): *"…recent investments in protective measures such as bulkheads or pump systems can make a substantial difference to community-level flood severity, as seen, for example, in West Wildwood, New Jersey. Upgrades to bulkheads made there in 2016 substantially reduced the extent and frequency of flooding the community experiences"* (Spanger-Siegfried & al. 2017).

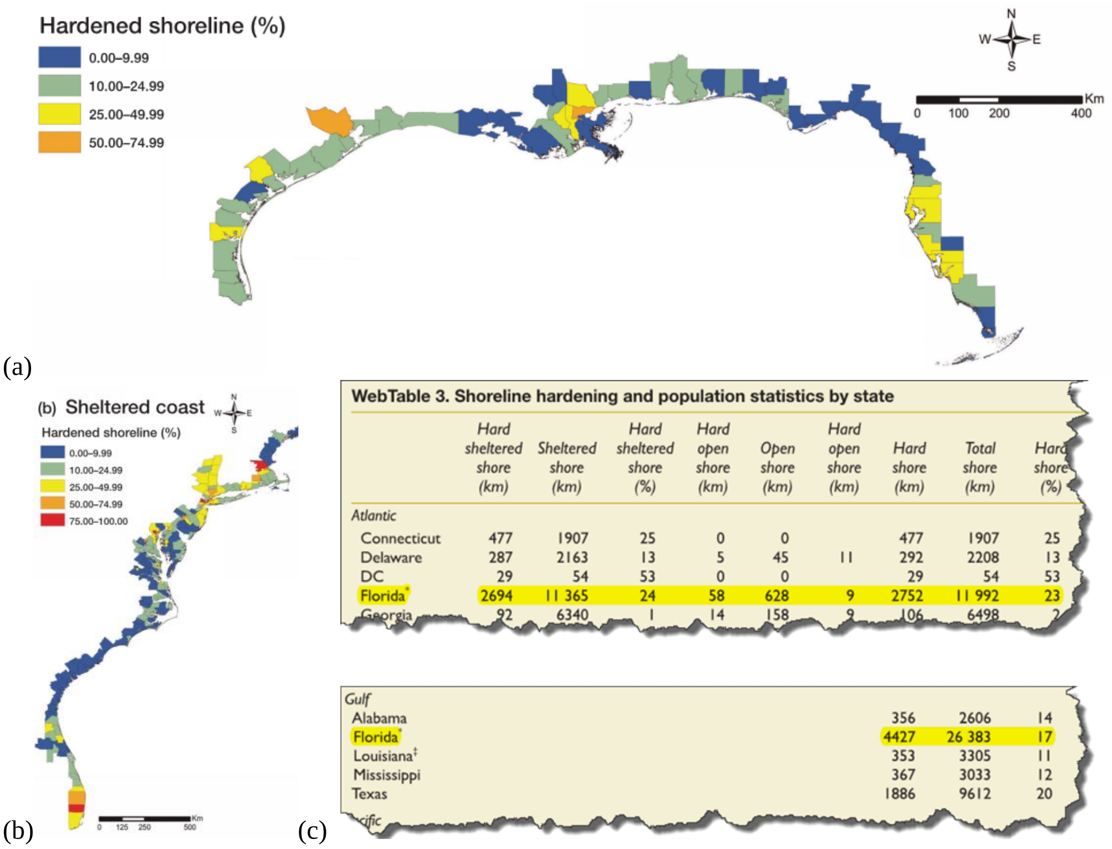
The major share of the liability will be borne by private property owners since they own a substantial majority of these structures along Florida's waterways. Some municipalities have established maintenance authorities using local taxation or assessment mechanisms to share the burden among stakeholders by funding continuous maintenance and periodic replacement. The City of Punta Gorda and Charlotte County are both good examples of this strategy (City of Punta Gorda 2018; Charlotte County 2016). Other communities in Florida are addressing rehabilitation and SLR by providing standards and regulations, including raising minimum seawall elevations (City of Miami 2012; City of Miami Beach 2016; City of Fort Lauderdale 2016)

There are many challenges in providing an economical and sustainable solution to the seawall MMR legacy, and divergent goals of the different owners. Generally, government owned structures are designed for a longer life, with authorities willing to invest more money for this long-term payoff. Private property owners often have more limited resources and may be inclined to pursue shorter-term economical solutions, especially if they do not foresee keeping their real estate in perpetuity.

A solution to the MRR challenge would certainly benefit from a coordinated approach and could effectively be guided by the collaboration of government, academia and private commercial industry, considering a holistic approach. The limited understanding of the durability limits and ecological impacts of these legacy systems at the initial installation time also necessitates a different approach moving forward for sustainable investment of future resources. A research collaboration project (SEACON), which will be discussed at the end of this paper, explored alternative approaches which may be applied as part of one possible solution to this pending durability and perhaps sustainability challenge. Adaption and ecological improvements of these systems is also under exploration by a collaborative team of multi-discipline engineers and ecologist under the proposed SEAHIVE project. This project is currently under consideration for an NCHRP Highway IDEA project (NCHRP 2018), with preliminary work underway at the University of Miami and Florida Department of Transportation, and commitments from other academic and municipal resources. The project involves the synergist effects of structural redundancy, wave attenuation, shoreline softening and biological habitat expansion over traditional solutions. It is expected to be another tool in the spectrum of Living Shoreline solutions that are gaining acceptance in the US.

3 Quantifying the shoreline legacy

Geographic Information Systems (GIS) can be leveraged to estimate the quantity of Florida shoreline hardened with concrete sheet piles based on the databased described in Gittman & al. (2015). The database files assembled by the National Oceanic and Atmospheric Administration (NOAA) for the Environmental Sensitivity Index (ESI) can be extracted and tabulated to quantify the hardened shoreline for open and sheltered shorelines along the US coastline. Sheltered shorelines will typically have sheet pile bulkhead armoring (either steel, aluminum, vinyl, timber or concrete), while open shorelines will typically only be concrete walls, rock revetment, or occasionally steel sheet piles. Figure 2 shows color coded maps of the US Atlantic and Gulf coasts based on the percentage of hardened shoreline for each county.



WebTable 3. Shoreline hardening and population statistics by state

	Hard sheltered shore (km)	Sheltered shore (km)	Hard sheltered shore (%)	Hard open shore (km)	Open shore (km)	Hard open shore (km)	Hard shore (km)	Total shore (km)	Hard shore (%)
Atlantic									
Connecticut	477	1907	25	0	0		477	1907	25
Delaware	287	2163	13	5	45	11	292	2208	13
DC	29	54	53	0	0		29	54	53
Florida*	2694	11 365	24	58	628	9	2752	11 992	23
Georgia	92	6340	1	14	158	9	106	6498	2
Gulf									
Alabama							356	2606	14
Florida*							4427	26 383	17
Louisiana‡							353	3305	11
Mississippi							367	3033	12
Texas							1886	9612	20

Fig. 2 Shoreline hardening: (a) Gulf coast; (b) Atlantic coast; (c) Summary statistics highlighting Florida *(Gittman & al. 2015).*

Figure 3 shows a summary of the lengths and the relative significance of this historical shoreline hardening for the State of Florida compared to the rest of the country.

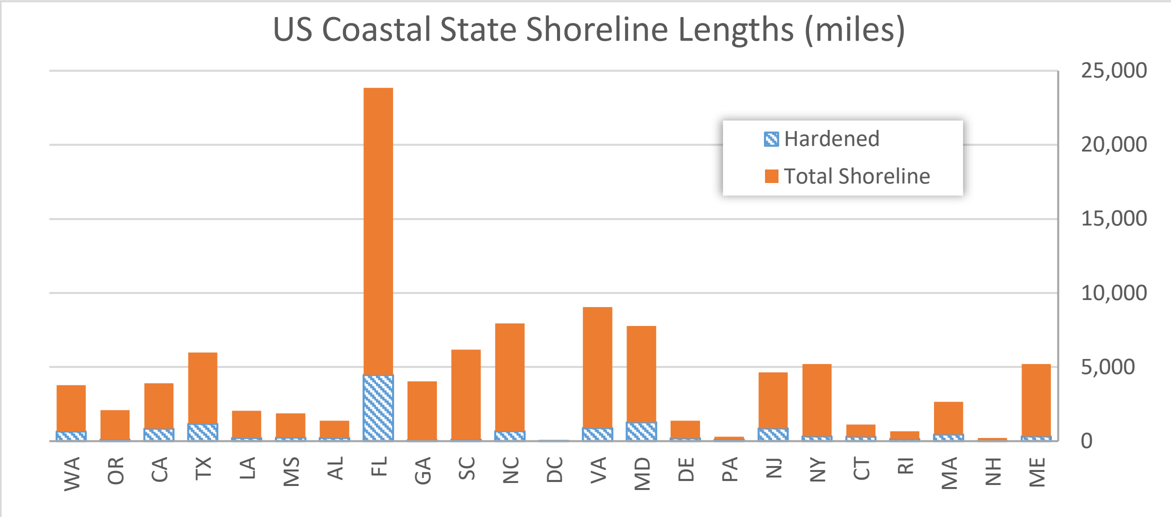


Fig. 3 Stacked bar graph showing total lengths of hardened shoreline and the significance of Florida shoreline compared to other states. *Louisiana (LA) does not include extensive bayou shoreline length. *(NOAA source data reported in Gittman & al. (2015))*

Several organizations have also developed GIS applications using the shape files that NOAA has assembled along with other data for visualizing a variety of interests. The Nature Conservancy (TNC) has combined these files in various tools with other biological information to look primarily at threats to coastal species and ecosystems, but they are still very useful for engineers and planners to investigate the extent of coastline armoring (TNC 2007).

Figure 4 is a screen shot from the TNC GIS mapping tool showing the seawall-bulkheads surrounding Fisher Island, Miami Beach. The layer is labeled *Estuarine Shoreline Suitability for Resilience Projects*, and comes from the NOAA ESI geodatabase shape files (NOAA 2016) as shoreline element Type 1B (exposed, solid, man-made structures) and Type 8 (sheltered rocky shores/seawalls/vegetated banks, solid man-made structures).

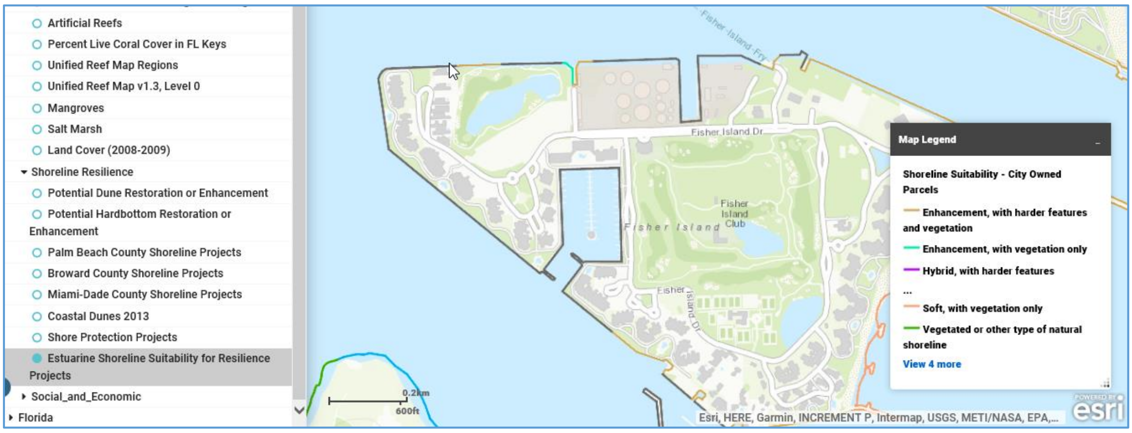


Fig. 4 Example GIS tool display using NOAA assembled shape files for shoreline hardening identification (Courtesy TNC, Coastal Resilience program)

The FDOT Office of Policy & Planning engage the University of Florida from 2012-2017 to develop a refine SLR and storm surge planning tool to investigate vulnerability of public infrastructure (Goodison & Thomas 2015). There have also been other similar efforts pursued in response to FHWA's initiative looking in transportation system resiliency and directive Order 5520 (FHWA 2014). Some completed projects include: NCHRP Report 750 - Climate Change, Extreme weather Events and the Highway System - Practitioner's Guide and Research Report (Meyers & al. 2014); Hillsborough County Vulnerability Assessment and Adaptation Pilot Project (Hillsborough County MPO 2014); and South Florida Climate Change Vulnerability Assessment and Adaptation Pilot Project (Broward MPO 2015), both of which were part of the 19 pilot projects around the US (FHWA 2016).

Table 1 provides a summary of shoreline hardening by county for both Atlantic and Gulf coasts in Florida, with temperature coded coloring based on the state totals. Gulf coast data is not broken into open and sheltered, but can generally be classified as sheltered compared to the Atlantic coast. It is estimated by the authors that more than 80% of this hardening consists of concrete sheet piling for an approximate total of 3,600 shoreline miles (5,800 km). The "Housing Density" and "Gross Domestic Product" columns are included to show the demographic trends and relative degree of potential collateral damage that could occur when these systems are breached or fail during storm events.

Table 1. Shoreline Hardening in Florida

County	State	Coast	Shore Type	Hard Shore (km)	Total Shore (km)	Hard Shore (%)	Housing Density (2010)	Gross Product (US$)	Domestic 2010
ATLANTIC COAST									
Brevard	Florida	Atlantic	Open	4	115	3.7	102	$	21,105,266,198
Broward	Florida	Atlantic	Open	2	39	6.0	260	$	76,109,761,398
Duval	Florida	Atlantic	Open	1	27	4.8	194	$	49,925,625,490
Flagler	Florida	Atlantic	Open	1	29	3.5	39	$	1,420,220,587
Indian	Florida	Atlantic	Open	1	36	1.7	59	$	4,073,674,416
Martin	Florida	Atlantic	Open	1	35	1.7	54	$	5,471,911,647
Miami-	Florida	Atlantic	Open	7	67	11.1	196	$	110,844,000,000
Nassau	Florida	Atlantic	Open	2	21	7.1	21	$	1,677,985,095
Palm	Florida	Atlantic	Open	9	74	11.7	130	$	57,324,794,060
St.	Florida	Atlantic	Open	3	66	5.2	145	$	4,992,345,230
St.	Florida	Atlantic	Open	2	36	5.6	122	$	5,545,737,425
Volusia	Florida	Atlantic	Open	25	81	30.3	89	$	12,862,432,071
Brevard	Florida	Atlantic	Sheltered	305	1316	23.2	102	$	7,764,270,511
Broward	Florida	Atlantic	Sheltered	578	675	85.6	260	$	1,452,317,020
Clay	Florida	Atlantic	Sheltered	62	210	29.4	48	$	4,797,447,921
Duval	Florida	Atlantic	Sheltered	182	1590	11.5	194	$	59,932,607,798
Flagler	Florida	Atlantic	Sheltered	107	335	32.1	39	$	3,921,783,874
Indian	Florida	Atlantic	Sheltered	70	407	17.3	59	$	585,892,965
Lake	Florida	Atlantic	Sheltered	25	193	13.0	59	$	21,105,266,198
Marion	Florida	Atlantic	Sheltered	4	71	6.3	40	$	19,180,076,071
Martin	Florida	Atlantic	Sheltered	132	351	37.7	54	$	1,225,639,608
Miami-	Florida	Atlantic	Sheltered	448	1253	35.7	196	$	25,283,362,876
Nassau	Florida	Atlantic	Sheltered	16	828	2.0	21	$	76,109,761,398
Palm	Florida	Atlantic	Sheltered	337	513	65.6	130	$	2,553,925,240
Putnam	Florida	Atlantic	Sheltered	1	530	20.7	20	$	454,320,043
Seminol	Florida	Atlantic	Sheltered	1	320	5.3	81	$	27,325,708,430
St.	Florida	Atlantic	Sheltered	1	737	14.0	145	$	23,318,243,157
St.	Florida	Atlantic	Sheltered	5	323	16.5	122	$	2,256,093,139
Volusia	Florida	Atlantic	Sheltered	1	1712	8.4	89	$	1,282,140,654
GULF OF MEXICO									
Bay	Florida	Gulf	Total	1	960	10.5	50	$	6,377,185,018
Charlott	Florida	Gulf	Total	5	1585	32.8	56	$	3,319,413,944
Citrus	Florida	Gulf	Total	3	2030	1.5	52	$	2,868,183,997
Collier	Florida	Gulf	Total	3	2335	16.2	38	$	11,691,137,192

Desoto	Florida	Gulf	Total	0	153	0.0	9	$	671,674,528
Dixie	Florida	Gulf	Total	5	402	1.3	5	$	174,291,007
Escambi	Florida	Gulf	Total	7	448	17.3	80	$	11,239,836,377
Franklin	Florida	Gulf	Total	4	1190	3.8	6	$	234,573,338
Gulf	Florida	Gulf	Total	8	481	1.7	6	$	300,004,976
Hernand	Florida	Gulf	Total	7	604	11.8	68	$	2,846,641,423
Hillsbor	Florida	Gulf	Total	2	987	25.9	197	$	65,174,508,345
Jefferso	Florida	Gulf	Total	0	89	0.0	4	$	185,744,806
Lee	Florida	Gulf	Total	1	2566	40.7	178	$	18,557,838,396
Levy	Florida	Gulf	Total	1	1351	1.3	7	$	589,708,087
Manatee	Florida	Gulf	Total	1	772	23.9	90	$	9,325,958,141
Monroe	Florida	Gulf	Total	4	5192	8.7	20	$	3,350,954,155
Okaloos	Florida	Gulf	Total	6	278	24.6	38	$	7,379,414,528
Pasco	Florida	Gulf	Total	1	451	35.9	119	$	8,033,482,322
Pinellas	Florida	Gulf	Total	5	1226	46.0	695	$	40,101,267,822
Santa	Florida	Gulf	Total	6	731	9.3	34	$	2,560,364,617
Sarasota	Florida	Gulf	Total	3	764	41.0	93	$	13,131,833,644
Taylor	Florida	Gulf	Total	9	445	2.1	4	$	615,145,073
Wakulla	Florida	Gulf	Total	2	798	2.5	8	$	458,859,651
Walton	Florida	Gulf	Total	3	485	6.6	16	$	1,446,014,036
Washin	Florida	Gulf	Total	0	59	0.0	7	$	458,519,336
Total (km)			**km**	**7,171**	**38,372**	**18.7%**		**$**	**840,995,165,279**
Total (miles)			**miles**	**4,460**	**23,843**	**18.7%**			
Open (km)			**km**	**58**	**626**	**9.3%**			
Open (miles)			**miles**	**36**	**389**	**9.3%**			
Sheltered (km)			**km**	**7,119**	**37,746**	**18.9%**			
Sheltered			**miles**	**4,424**	**23,454**	**18.9%**			

The authors estimated the asset replacement cost of the concrete seawall-bulkheads at $21Billion (present day cost), which is 2.5% of the 2010 GDP for the coastal counties listed in Table 1. This value is based on FDOT's 5-year statewide combined average unit rate of $61 per square foot (FDOT 2016a) and assumes an exposed wall height of 7-feet and buried length of 11-feet. This replacement cost does not include any additional costs associated with design and permitting, mobilization of construction equipment, removal of existing structures, additional toe revetment (such as rubble riprap), environmental controls and restoration of landscaping, sidewalks or railings.

4 Examples of seawall-bulkhead installations in Florida

RC seawall-bulkheads were used extensively in Florida to create and protect land with valuable waterfront views usually in combination with dredge-and-fill operations. Well known examples include Brickell Key, Fisher Island and Key Biscayne in Miami (Figure 5); Marco Island, Charlotte Harbor and Punta Gorda Isles in southwest Florida (Figure 6); Davis Island and Pinellas county barrier islands in the Tampa Bay region (Figures 1 & 7).

(a) 
(b) 
(c) 

(d) (e) (f)


Fig. 5 Examples of RC seawall-bulkhead usage for land development on Florida's east coast: (a) Brickell Key, 1928 (b) Fisher Island, 1920's (c) Key Biscayne [1], 1950's (d) Brickell Key [2], 2011 (e) Fisher Island, 2015 (f) Key Biscayne, 2015.

[1] *Image courtesy of Mackel Company.*

[2] *Photograph courtesy of Island Marine-Panoramio*

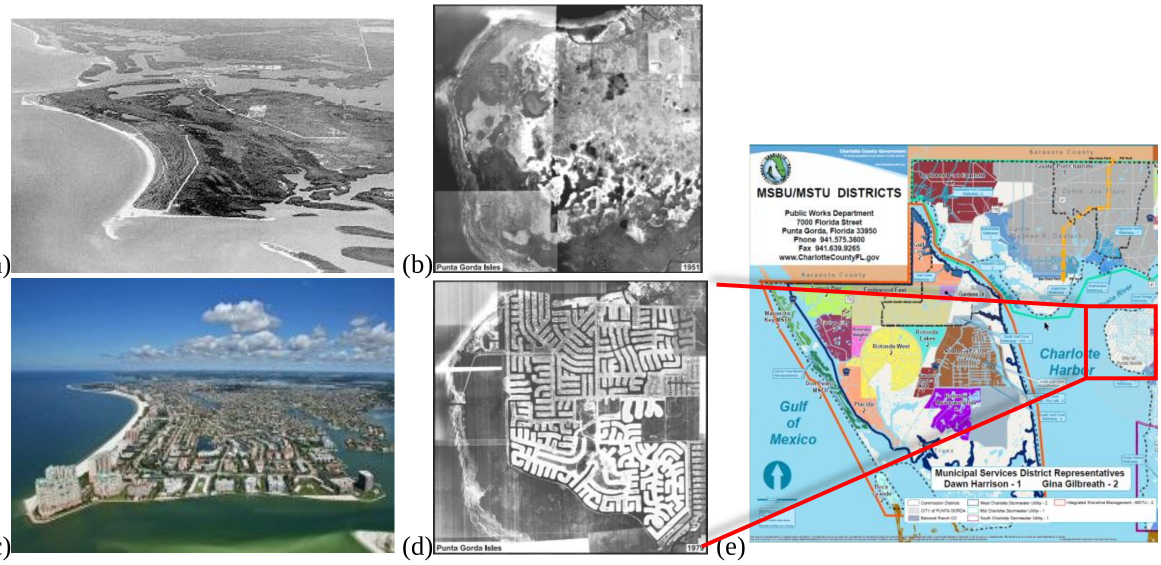
(a) (b) (c) (d) (e)


Fig. 6 Examples of RC & PC seawall-bulkhead usage for land development on Florida's gulf coast: (a) Marco Island [1], 1964; (b) Punta Gorda Isle, 1951; (c) Marco Island [2], 1969 Master Plan; (d) Punta Gorda Isles [3], 1979; (e) Charlotte county MSBU-Waterway Districts [4], 2015.

[1] *Photographs courtesy of Marco Island Chamber of Commerce.*

[2] *Photograph courtesy of Island Realty Marco.*

[3] *Map courtesy of Punta Gorda Isles Canal Maintenance District.*

[4] *Map courtesy of Charlotte County Public Works Department.*

(a) (b) (c)


Fig. 7 Davis Island, Tampa Bay reclamation using RC seawall-bulkheads: (a) 1924; (b) 1925; (c) 1926. *(Photographs courtesy of the Burgert Bros.)*

5 The sea level rise challenge

Given the historical trend in SLR and the recorded acceleration since the early 1990's (Meyers & al. 2014) the authors strongly recommend consideration of adaptive features. Projections of future SLR vary widely, so it is prudent to pursue a cost-effective strategy for adaptive features enabling raising seawall-bulkheads in the future to address challenges associated with SLR and nuisance flooding which currently manifest during king tide and storm surge events in southeastern Florida cities. The Intergovernmental Panel on Climate Change (IPCC) Working Group II, also expressed the task in the following way: *"developing adaptation responses [to climate change] requires a long, interdisciplinary dialogue between researchers and stakeholders, with substantial changes in institutions and infrastructure"* (Field & al. 2007).

Four counties and many associated cities in Florida under a regional climate change agreement (Southeast Florida Regional Climate Change Compact 2009) have already begun addressing adaption to SLR with interim measures. Much public debate, collaboration and development work for a longer-term strategy still lies ahead. *"South Florida, inclusive of Miami-Dade County (MDC), ranks as one of the world's most vulnerable urban regions in terms of assets exposed to coastal flooding exacerbated by sea level rise and violent storms"* (FIU 2015), and *"…no region in the country has tried to tackle climate change on the same scale as what the four South Florida counties propose, according to Ron Sims, former secretary of U.S. Housing and Urban Development."* (Reid 2012).

Southeast Florida has been specifically identified by the Union of Concerned Scientists, and two US federal agencies as extremely susceptible to the effects of SLR (Spanger-Siegfried & al. 2017; NOAA 2017; USACE 2013). Following are examples of actions related to seawall-bulkheads recently taken separately by several cities located within the counties participating in the Southeast Florida Regional Climate Change Compact, and some other related institutions:

5.1 City of Miami

The City of Miami revised its building ordinance, raising the minimum height of the seawalls to 6.0 feet NGVD for locations south of the Rickenbacker Causeway (City of Miami 2012). Mayor Regalado also requested $192M to combat the effects of SLR and associated flooding in his proposed budget release July 19, 2017 (Borge 2017).

5.2 City of Miami Beach

The City of Miami Beach recently revised its building ordinances raising the minimum height of the finished floor elevation to 3.0 feet above the base flood elevation and added the following SLR projection for planning purposes*:*

i. **Short term**, by 2030, sea level is projected to rise 6 to 10 inches above 1992 mean sea level,
ii. **Medium term**, by 2060, sea level is projected to rise 14 to 34 inches above 1992 mean sea level,
iii. **Long term**, by 2100, sea level is projected to rise 31 to 81 inches above 1992 mean sea level.

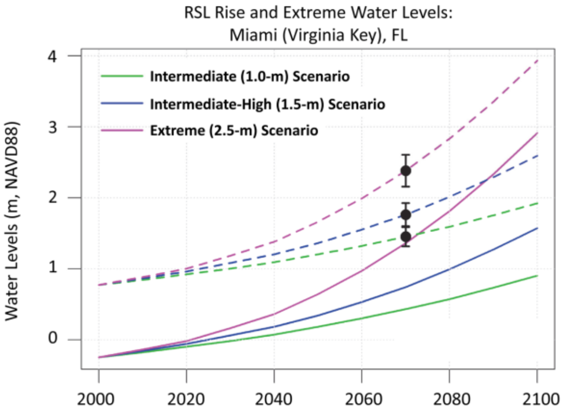
The city also updated their Public Works Manual, raising the minimum seawall elevation from 3.2 feet to 5.7 feet NAVD (7.26 feet NVGD) which is based on a Category 1 Hurricane or 50-year storm event (City of Miami Beach 2016).

5.3 City of Fort Lauderdale

In late September and early October 2015, the City experienced unprecedented flooding during the seasonal King tides. The City subsequently development regulations establishing a seawall height at 3.9 feet NAVD (5.5 feet NGVD) minimum, 5.0 feet NAVD (6.6 feet NGVD) maximum, but added: "*Property owners choosing to construct seawalls at less than 5.0 feet NAVD88 are strongly encouraged to have the foundation designed to accommodate a future seawall height extension up to a minimum elevation of 5.0 feet NAVD88."* (City of Fort Lauderdale 2016).

5.4 American Institute of Architects Florida

Last year the Florida Association of the American Institute of Architects issued a policy position statement on future sea level rise, in a press release (AIA Florida 2017). Additionally, the former Chapter president, Andrew Hayes, chair of the AIA Florida Strategic Council had previously stated to reporters *“Our members should plan for three feet of sea level rise (SLR), in dealing with clients, municipal building codes and related professionals, such as engineers”* (Coastal News Today 2016).

(a)


(b)


Fig. 8 (a) NOAA (2017) projections under Global Mean Sea Level rise scenarios (solid curves), and how the water level height increases with a 1% annual chance of occurring (dashed lines) *pg.40;* (b) Hurricane Sandy damage along State Highway A1A in Fort Lauderdale *(Photo: Susan Stocker, Sun Sentinel, 2012)*

5.5 US Federal Agencies

Federal agencies are also providing tools and data for local communities to encourage informed policy debate, strategic planning and community response. The following extract is from NOAA (2017) *“…we find at most locations examined (90 cities along the U.S. coastline outside of Alaska) that with only about 0.35 m (<14 inches) of local RSL rise, annual frequencies of such disruptive/damaging flooding will increase 25-fold by… 2080, 2060, 2040 and 2030 under the Low, Intermediate-Low, Intermediate and Intermediate High subset of scenarios, respectively.”*

The federal Coastal Zone Management Act as amended by Public Law 109-58 - Energy Policy Act of 2005, provides funding to states that adopt plans for managing development in coastal areas, requires coastal states to plan for the effects of climate change: *“Because global warming may result in a substantial sea level rise with serious adverse effects in the coastal zone, coastal states must anticipate and plan for such an occurrence"* (US Congress 1972a).

5.6 Non-Profit Groups and Developers

Ten years ago, the non-profit group (NPG) Center for Progressive Reform published proposed legislative changes to the 1972 Clean Water Act: A Blueprint for Reform (Andreen & Jones 2008) attempting to address SLR, and even private land developers are now seeking government action and leadership on SLR: Alan Faena, the Argentinian developer behind the new leisure and cultural quarter bearing his name in Miami Beach expressed in a recent interview that climate change is *"a problem... We try to do our best, but that's something that should come from government and NPGs"* (Fairs 2016).

6 Florida seawall-bulkhead standards

A review of records from the Florida’s State Road Department, which was later reorganized into the Florida Department of Transportation (FDOT), provides the general progression in the development of concrete sheet pile bulkhead systems. Figure 9 shows some typical early precast bulkhead details, while Table 2 provides a complete list of archived standards available from the FDOT State Structures Design Office.

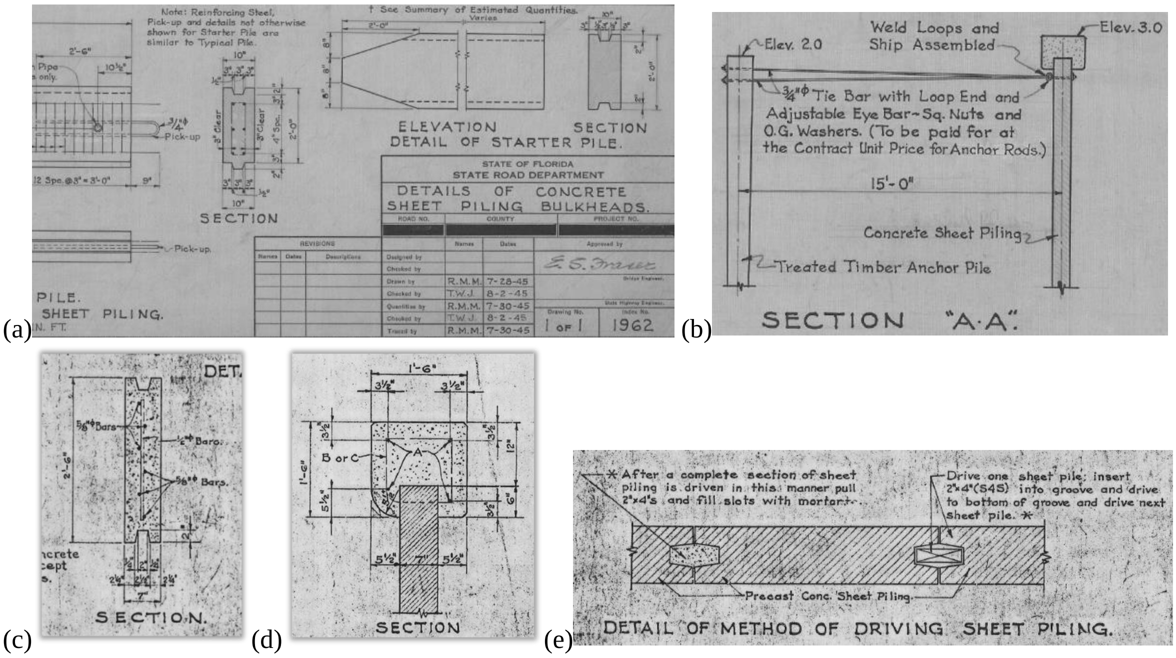


Fig. 9 Historical FDOT precast sheet pile details: (a) & (b) *Index 1962*, 1945; (c) Section thru RC sheet pile; (d) Section thru bulkhead cap; (e) Detail showing joint sealing and alignment technique, *Index 2039*, 1946.

Table 2. Historical FDOT Bulkhead Standard Drawings (1938 - 1970)

Year	Index	Description
1938	1442	Standard H-Pile Bent ~ 24'-0" Roadway. Treated Timber Bulkheads
1939	1501	Bulkhead Details [Timber] for Widening Existing Bridges on the Tamiami Trail
1942	1786	Details of Bulkheads and Splash Walls [Timber]
1942	1819	Standard Treated Timber Bulkhead-26 Ft. Roadway
1945	1943	Standard Treated Timber Bulkhead
1945	1962	Details of Concrete Sheet Piling Bulkheads
1946	2039	Details of Concrete Sheet Piling Bulkhead
1948	2260	Details of Concrete Sheet Piling Bulkhead
1950	2467	Precast Concrete Sheet Piling
1951	2615	Concrete Bulkheads-15' Span-28' Roadway-2~3'-1" Sidewalks
1951	2686	Details of Special Sections Concrete Sheet Piling Bulkhead
1952	2692	Precast Cantilever Bulkhead
1953	3040	Concrete Sheet Piling Bulkheads
1954	3204	Concrete Bulkheads with Composite Master Piles and Precast Slabs
1954	3235	Concrete Sheet Piling Bulkhead
1954	3388	Precast Concrete Sheet Piling
1957	3904	Precast Concrete Sheet Piling
1957	3950	Precast Concrete Sheet Piling (10" x 30")
1957	4051	Concrete Sheet Piling Bulkhead Details
1960	4057	Precast Concrete Sheet Piling (8" x 30") [Prestressed]
1957	4059	Precast Concrete Sheet Piling (12"x30") [Prestressed]
1959	4513	Precast Concrete Sheet Piling (7" x 30") [Prestressed]
1961	7284	Precast Concrete Sheet Piling (8" X 30") [Prestressed]
1970	3950rev	Precast Concrete Sheet Piling (10" x 30") [Prestressed]

Concrete sheet piles were initially designed with mild steel reinforcing Grade 33 (230 MPa), and later Grade 40 (275 MPa). PC sheet pile standards were available by late 1957 under ***Index 4057***, and have changed very little in configuration since that time. The original PC sheet piles used 7/16-inch (11mm) dia. stress-relieved steel strands with a concrete pre-compression of approximately 800 psi (5.5 MPa), which was similar to the prestressing level used for standard bridge bearing piles in that time period. By 1970 the minimum prestressing force was increased to 1,000 psi (6.9 MPa) using ½-inch (13mm) or 0.6-inch (15mm) dia. low-relaxation 270 ksi (186 MPa) strand, similar to today's FDOT standards (***Index 6040***), and the supplemental reinforcing upgraded to Grade 60 (415 MPa).

Typically, the design of concrete sheet pile bulkheads are governed by Service Limit State (SLS) concrete tensile stresses with the goal of minimizing cracks which can lead to corrosion, concrete spalling and then progressive deterioration of the structure. The 20% increase in prestressing force after 1970 provided a 20% higher flexural capacity at the SLS. This enabled designers to accommodate taller retained fills or additional live load surcharge, and the associated challenges with handling and shipping longer sheet piles needed. Maximizing transportation and handling capacity is very beneficial in reducing constructability costs, especially considering the advantage of using a "single-point pick-up" at the end of the sheet pile during installation (Figure 10). In most cases however, the in-service soil pressure loading will govern the seawall-bulkhead wall design and is typically the first check in selecting a sheet pile size.

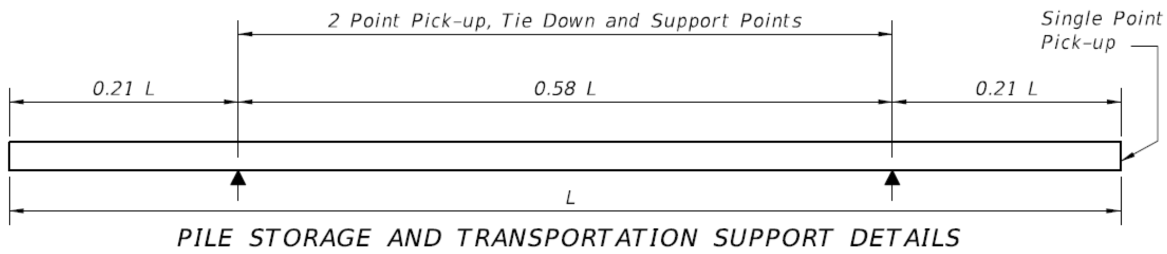


Fig. 10 Schematic of typical lifting points for handling, storing, transporting and installation of prestressed concrete sheet piles, FDOT *Index 6040*, 2017.

There are several other concrete bulkhead systems utilized in South Florida due to the shallow limestone geology in that region. The most common is the soldier pile and panel system, which is especially popular with the cities of Miami and Miami Beach. Figure 11 shows an active installation from a recent City of Miami Beach project. This project is part of a $400M initiative combating "sunny day" flooding from high tide and storm surge which pushes seawater into streets and neighborhoods, a trend the city expects to continue.

Several communities along Florida's southwest Gulf coast utilize a non-prestressed panel tie-back bulkhead system with its advantage of shallower installation depths. Many of these systems were damaged or failed during the 2017 hurricane season (City of Punta Gorda 2017). Neither of the south Florida systems was ever formally standardized by the FDOT and the details are outside the scope of this paper, however the FDOT is currently developing more durable alternatives by converting these systems to utilize corrosion-resistance materials such as stainless-steel, carbon and glass fiber-reinforced polymer strand and rebar, that can in-turn utilize more sustainable cement and concrete mixes (SEACON) without premature deterioration.

(a)  (b)

Fig. 11 (a) Miami Beach seawall-bulkhead and road raising project [1]; (b) North Miami Beach deteriorated seawall due to corrosion.

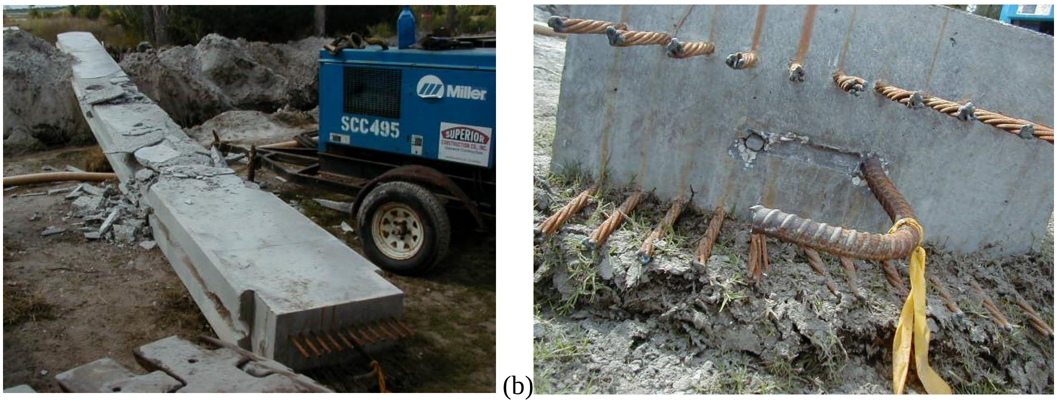
[1] *Photo courtesy of Bruce Mowry, former City Engineer.*

[2] *Photo courtesy of Joaquin Perez, Bolton Perez & Associates Consulting Engineers.*

Strategies for increasing the capacity of existing sheet pile standards to accommodate increased loading and lengths as a result of Extreme Weather and SLR, include: increasing pile thickness or stiffness; increasing prestress force; and increasing concrete tensile stress limits.

6.1 Increasing Pile Thickness

This strategy results in significant increase in the cross-sectional flexural modulus, since the surface tensile stress is a function of the square root of the section thickness. A disadvantage of this approach is that handling weight increases proportionally requiring larger crane capacities and shipping costs. Also the handling length capacity only increases slightly with thickness since the self-weight flexural stress is a function of the square of the sheet pile length. This approach was utilized by the first author for a bridge project in 2001 along State Road A1A over Ft George River Inlet (Nolan 2001). The exposed maximum wall height was 23-feet (7 m) in combination with tie-back anchors using a prestressed sheet pile section thickness of 16-inches (405mm). Figure 12 shows one of the heavier sheet piles that was dropped during installation, due to the unfamiliarity of the contractor with handling such large sections.

(a) 

(b)

Fig. 12 (a) 16-inch (400mm) thick concrete sheet pile dropped during installation; (b) fractured steel lifting loop due to mishandling heavier precast element.

6.2 *Increasing Prestressing Force*

This strategy is contingent on both the compression and tension design limits under the SLS. Balancing the degree of prestressing against these limits can maximize the benefits of the constituent materials strength properties. When designing to the ***AASHTO LRFD Bridge Design Specification, 8th edition*** (AASHTO 2017), tension stresses for prestressed concrete are limited to $\boldsymbol{0.0948\sqrt{f'_c}}$ *(ksi)* for marine "corrosive" environments using steel strands, while sustained concrete compression stress is limited to $\boldsymbol{0.4f'_c}$ *(ksi)*. In simplistic terms, a prestressing force at the average of these stresses provides the largest SLS capacity (balanced design), but not necessarily the most cost effective design.

The standard strength concrete FDOT specifies for prestressed products in marine (corrosive) environments is Class V (Special) – 6.0 ksi (41.4 MPa) 28-day strength. The long-term pre-compression stress for the SLS balanced design would be 1.3 ksi, which is only slightly higher (15%) than that 1.10 ksi 1.16 ksi (7.6 to 8.0 MPa) specified under ***Index 6040***. The disadvantage of increasing the prestressing force is that it usually requires additional strands, which add cost and can also result in additional end cracking from the higher prestress transfer forces, and may require additional confinement reinforcing.

6.3 *Increasing Flexural Tension Limits*

This strategy is based on the tolerance for the risk of concrete cracking and the resulting potential loss in durability. The AASHTO 2017 concrete tension stress limit of $0.0948\sqrt{f'_c}$ (ksi*)* is a well-established conservative value to avoid concrete cracking in flexural elements in corrosive environments. Other residual tensile stresses can also be present in the element due to concrete shrinkage, temperature gradient and/or structural system boundary restraint effects, which can potentially initiate premature cracking and may not be accounted for in the AASHTO 2017 imposed limit.

The use of corrosion-resistant prestressing and reinforcing materials allow for the potential increase in concrete flexural tension limits and eliminate concerns from unexpected cracking due residual stresses. For corrosion-resistant prestressing strand FDOT currently uses a concrete tension limit of $0.19\sqrt{f'_c}$ *(ksi)*, under ***Index 22440*** (FDOT 2016b). This is the traditional concrete tension stress limit for slightly-to-moderately aggressive environments using carbon-steel prestressing and reinforcing materials, but since the strand is not susceptible to corrosion, was conservatively adopted in the spirit of AASHTO 2017 *Table 5.9.2.3.2b-1 "…For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions"*.

With corrosion-resistant materials such as fiber-reinforced polymers (FRP) and 2205 stainless steel, it may even be possible to allow higher tension stresses with a greater tolerance for cracking, especially for locations were freeze-thaw is not a common occurrence. However, the resulting loss of stiffness and increased deflection effects need to be carefully evaluated.

7 Looking forward: Advancing the state-of-practice

This section presents a culmination of these efforts and a vision moving forward applicable to many coastal communities worldwide. As part of the FDOT's Transportation Innovation Challenge (TIC), fiber-reinforced polymer (FRP) composite and stainless-steel (SS) prestressed concrete standards were developed for broader deployment of bridge piling, seawall sheet piling and bulkhead cap construction and replacement.

Employment of corrosion-resistant reinforcement represents a widely recognized effective strategy to ensure long-term durability of reinforced concrete (RC) and prestressed concrete (PC) structures. FRP composites have proved to be a reliable non-metallic solution, able to provide both the required mechanical and corrosion resistant properties (Nanni 2000). FRP-RC infrastructure applications are currently spreading; conversely, FRP-PC is still considered an emerging state-of-the-art technology and standard design provisions are currently in final stages of development in the US (ACI 2011; Belarbi 2018).

Among composites, carbon FRP (CFRP) has historically been the preferred solution for PC applications, due to its good mechanical performance under sustained load, in addition to durability. The technology is already available and currently deployed in the construction of the Halls River Bridge in Homosassa, Florida (Rambo-Roddenberry & Gartman 2017; Rossini & al. 2018a).

7.1 SEACON

The FDOT collaborated with the University of Miami (UM) and Polytechnic Milano as part of the multinational Infravation-SEACON project (Bertola & al., 2017). The SEACON project seeks to enhance and refine the application of advancements in materials technology that can be effectively leveraged from durability, economic and sustainability aspects. Primarily focused on FRP composite and SS reinforcing in combination with seawater concrete and chloride contaminated recycled concrete aggregate, the SEACON research team is also partnered with industry producers (Acciaierie Valbruna, ATP, Buzzi Unicem & Owens Corning) and Italian transportation agency counterparts (Pavimental, part of Autostrade per l'Italia). Field demonstrations of real world structures utilizing these components have been constructed, documented and are under continual monitoring as part of both the SEACON project, and FDOT's TIC initiative.

7.2 Glass FRP Prestressed Concrete

The application of glass FRP (GFRP) to mild-prestress concrete sheet piles is currently under investigation (Rossini & al. 2018b). Glass fiber is an economical alternative to carbon fiber in applications that do not require a high level of concrete pre-compression. Limiting the amount of the initial prestressing could overcome most of the constructability issues observed with CFRP tendons which require complex and expensive anchors, while the reduced cost of glass makes it a competitive and durable alternative to standard steel strands.

An NCHRP Highway IDEA project (MILDGLASS) was funded in 2018 to further refine this technology. At the current prototype stage, the design of a 30x10-inch (762x254mm) sheet pile prestressed with two rows of eleven strands each is realistic. The concrete pre-compression would be limited to 700 psi (4.83 MPa) which is consistent with current FDOT corrosion-resistance prestressed concrete sheet pile standards (FDOT 2016b). The proposed GFRP-PC solution shows the potential of becoming an economical alternative to steel-PC in the near future. The goal is to eventually allow for the design of 30x12 inch (762x305 mm) sheet piles using two rows of 10 strands each (Figure 13). Parallel research within the SEACON project is confirming the excellent GFRP durability performance in marine environments (Khatibmasjedi & al. 2017*)*.

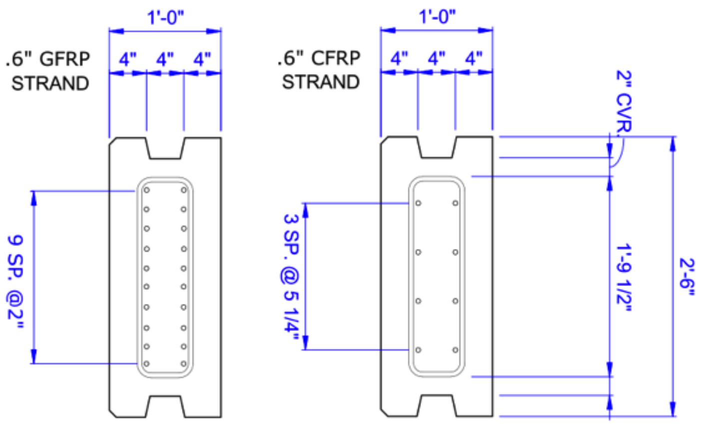


Fig. 13 (a) GFRP-PC sheet pile concept (b) CFRP-PC sheet pile design for Halls River Bridge project.

The initial research effort focused on tensile testing of GFRP 7-cord twisted strand prototypes anchored to the cross-heads of the testing frame with conventional steel prestress chucks (Figure 14 and 15). The aim was to verify prototype compatibility with construction techniques traditionally applied to steel-PC.

The tested samples differ from each other in terms of production quality. It is acknowledged how the inevitable presence of defects is related to the prototypical nature of the product and shall not be deemed representative of a large-scaled quality-controlled production. The variability has been initially addressed in the statistical data set, by separating defective samples from quality-controlled counterparts.

Samples have been tensile tested up to failure in load control to better reproduce the actual jacking load applied on site. Two different failure modes have been experienced, happening at the same level of applied load, either local slipping or local rupture at one of the chucked ends. Results are showing a quality-controlled production able to guarantee an instantaneous jacking stress of more than 12 kip (45% of nominal ultimate capacity). Creep and relaxation tests are currently undergoing and are confirming the ability of the material to sustain a sufficient level of load, without experiencing damages or excessive losses.

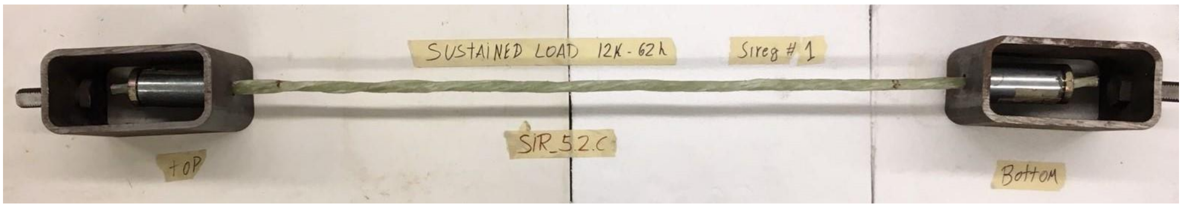


Fig. 14 GFRP strand prototype with steel chucks and cross-heads, ready for test.

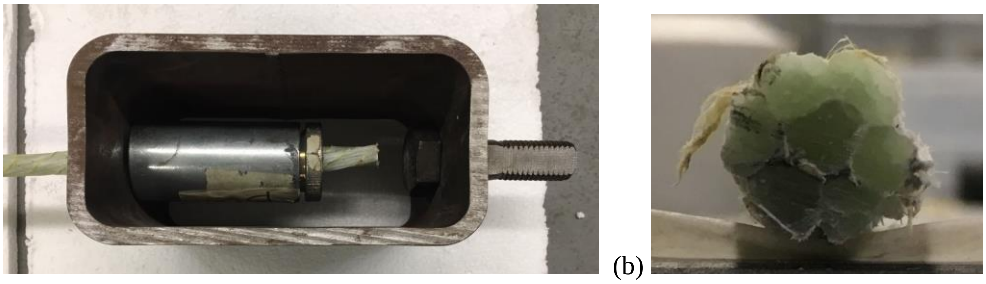
(a)  (b)

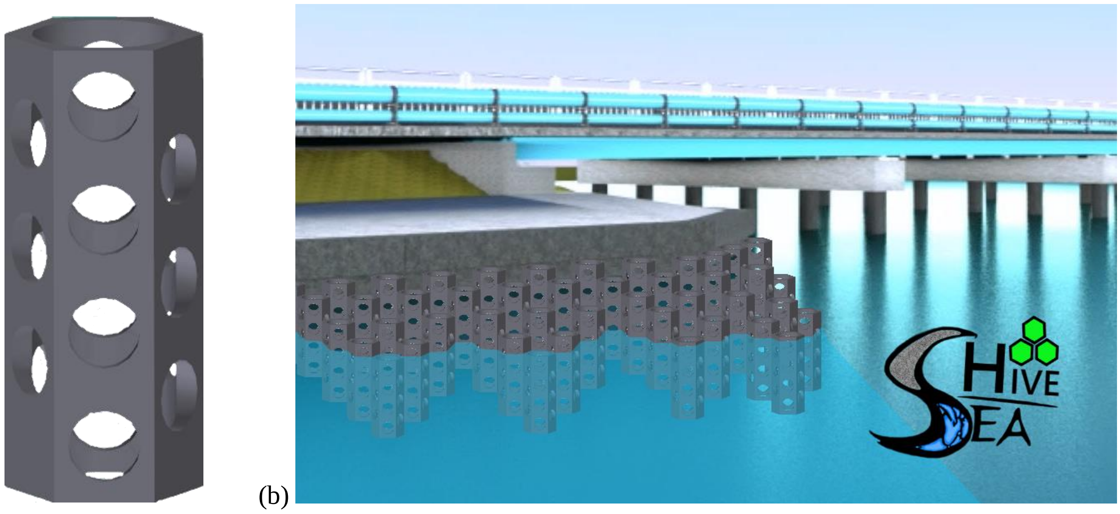
Fig. 15 (a) Steel chuck installed on strand prototype, within frame cross-heads; (b) Cross section of GFRP 7-cord twisted strand.

7.3 SEAHIVE

Another collaboration between the University of Miami and FDOT involves the development of engineer precast concrete revetment (Rhode-Barbarigos 2018). The proposed project focuses on developing a novel efficient, cost-effective and ecofriendly modular shoreline protection system based on perforated precast hexagonal tubes (hextubes). The hextubes can be produced with traditional pipe or precast/prestress techniques. Typically fibre-reinforced, with low alkalinity concrete and/or reduced use of portland cement, and no chloride limits (sea-concrete) concrete. Non-corrosive reinforcing (GFRP, BFRP or CFRP) is proposed for deep stacked installations or high wave energy locations and to improve compatibility with marine life and sustainability. The size and shape of the hextubes will be adapted to increase wave energy dissipation. The outer form of the hextubes has a faceted surface, which will facilitate stacking and interlocking stability, while perforations in these faces will provide passage for flow of water under surging or breaking waves, and allow for dissipation of wave energy (see Figure 16). In a system configuration, voids can also form interconnected channels providing habitat and protection for marine life.

The SEAHIVE system could be applied to the protection of existing sheet pile and bulkhead solutions, or for the replacement of deficient shoreline protection systems. It can be also coupled with non-corrosive FRP-PC sheet piles and seawall-bulkheads for a complete resilient replacement package.

The current practice of stacking natural rock material or non-organized artificial elements has proved ineffective as shown by many bulkhead collapses during recent hurricane events (City of Punta Gorda 2017). Granite boulders and rubble usually need to be quarried in mountainous areas and shipped to coastal areas, resulting in a significant transportation cost component. Traditionally, artificial concrete alternatives are also heavy, difficult to handle, transport and install, often aesthetically monolithic and potential environmentally intrusive.



(a) (b)

Fig. 16 (a) Prototype hextube; (b) Typical revetment application for bulkhead-seawall.

8 Conclusion

The liability represented by the aging reinforced concrete seawall-bulkheads in Florida was discussed in detail and the concurring challenges presented by extreme weather and SLR were explored. Actions currently undertaken by government agencies and research institutions have been presented, with reference to the SEACON, MILDGLASS, and SEAHIVE research projects. Durable alternatives to traditional steel reinforcing and prestressing are now available for use in Florida. Promising early results from the MILDGLASS project are expected to provide low-cost, resilient alternatives, in addition to the sustainable material systems identified and verified under the SEACON project. Work under the SEAHIVE project will provide ecofriendly resilience and adaptation products to meet the emerging challenges, with potential improvements in wave attenuation, bio-habitat and sustainable structural materials.

Acknowledgements

The authors gratefully acknowledge the financial support from (1) “Sustainable concrete using seawater, salt-contaminated aggregates, and non-corrosive reinforcement” Infravation, 31109806.005-SEACON, Awarded, 10/01/15-3/31/18; and, (2) Zegna Foundation for supporting the second author’s research activity in the field of composite materials for infrastructural applications. The opinions expressed and conclusions in this paper are those of the authors and not necessarily those of the Florida Department of Transportation or the University of Miami.

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TRB Annual Meeting, (Washington DC.) January, 2018

Session 679 - Wednesday 1/9/2018, 3:45-5:30pm

Seawalls, *SEACON*, and Sustainability in the Sunshine State

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FDOT State Structures Design Office

Marco Rossini, M.S.

University of Miami, College of Engineering

Key Topics

1. What is the problem?
2. Are Composites the solution?
3. History of Seawall Development in Florida
4. Quantifying the Shoreline Legacy
5. New Challenges *– SLR, Extreme Weather, Sustainability, Increased Durability Expectations*
6. New Solutions *– SEACON, GFRP-PC, BFRP*
7. Florida Project Examples

What is the Problem?

Need for New Solutions for Corrosion Durability and Sustainability

Avoiding corrosion *“concrete cancer”*

- GFRP or SS rebar
- CFRP or HSSS prestressing strand

i. Cost-Benefit Analysis, LCA/LCC;

ii. Durability = Long Service Life;

iii. Challenges & Mitigating Risks

- Acquisition Cost
- New Material Systems;
- Limited suppliers/competition;
- Unfamiliar design criteria;
- Unfamiliar construction practices.

New and Old Seven-Mile-Bridge, (Florida Keys)

Courtney Campbell Causeway, seawall (Tampa Bay)

Gandy Blvd. seawall, (Tampa Bay)

What is the Problem?

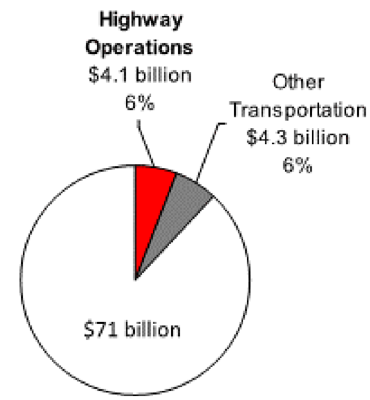
Example Costs of Corrosion ($$)

EXAMPLE:

Transportation- 12% of Florida's Budget

- Large integrated investment in state bridges. ~6,000 bridges.

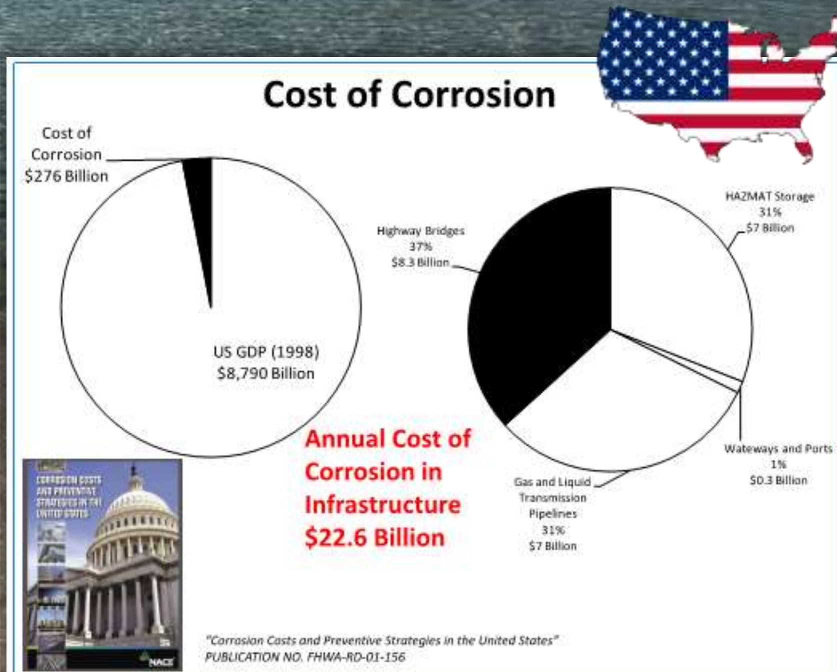
  1/2 in aggressive marine service.
- ~ $300 million per year spent on bridge construction. Additional yearly costs for maintenance.
- 75-year design life - potential huge cost in life reduction due to corrosion.
- *Need to improve design to control corrosion, develop tools to assess future performance to decide on best design and rehab alternatives, and assess need for future maintenance.*



source: The People's Budget. www.ebudget.state.fl.us

FDOT

Chart: FY 2012-2013 http://www.floridafirstbudget.com/ (FY 2015-16: Total = $78B, Hwy.Op. = $5.6B, Other = $4.4) from TRB webinar "[Controlling Corrosion of Infrastructure Systems](#)" – K. Lau & M. O'Reilly, August 2016.



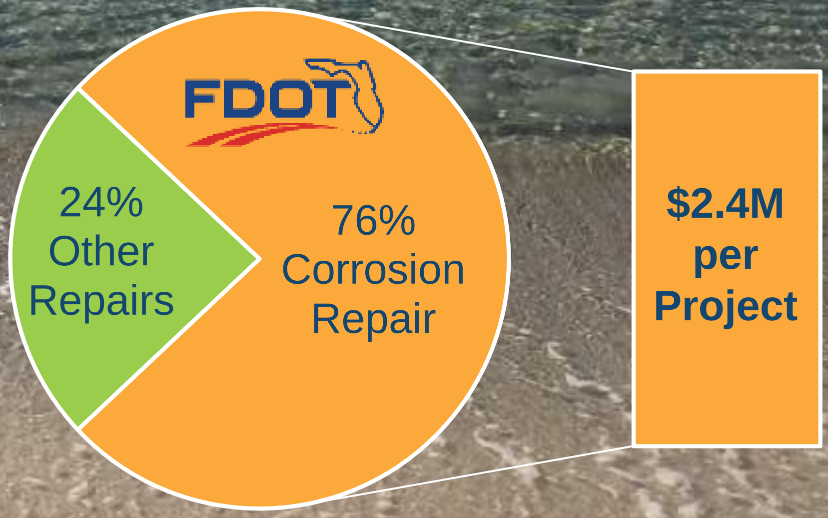
https://www.nace.org/uploadedFiles/Publications/ccsupp.pdf

What is the Problem?

Example Costs of Corrosion (District 7- Tampa Bay)

Repair cost of bridges in District 7 *(FY 2002/03 to 2012/13)*

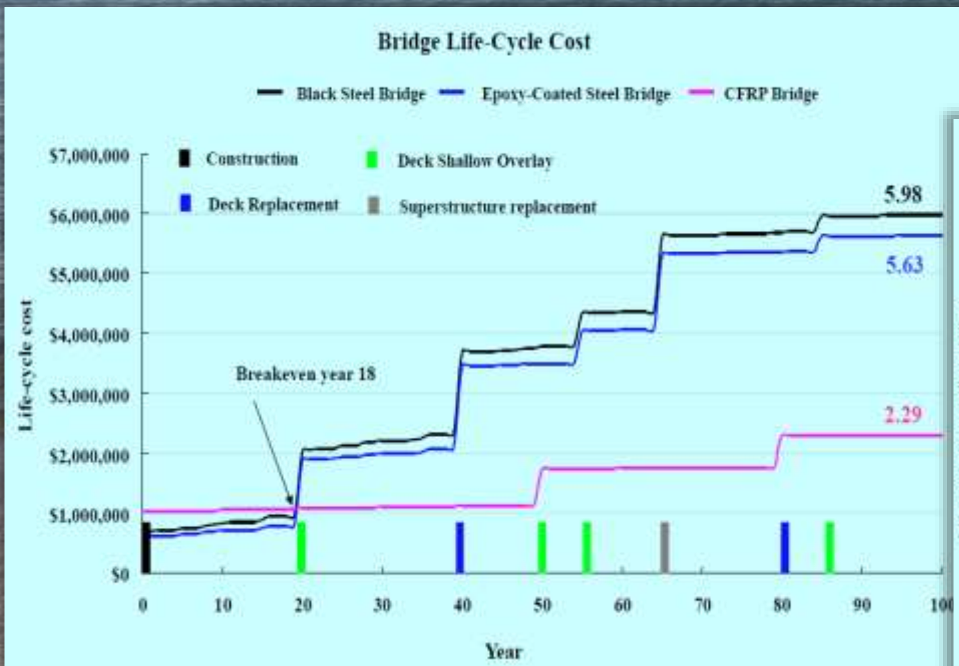
- 54 Bridge projects studied over ten year period
- (20 Steel Bridges and 34 Concrete Bridges)



Source: FDOT D7 District Structures Maintenance Office & T.Y. Lin

Are Composites the Solution?

Bridge Life-cycle Cost (surrogate for seawalls)



*Source: **Ohio Bridge Design Conference presentation**, "New Generation of Sustainable CFRP Prestressed Concrete Highway Bridges", slides 25-26. (Dr. Nabil Grace, 2014)*

Are Composites the Solution?

Service Life Enhancement thru Durability:

- **50 years** under ***AASHTO Standard Specification for Highway Bridges*** (1970's??? - 2002)
- **75 years** under ***AASHTO LRFD Bridge Design Specification*** (1994 – present)
- **100 years +**, ***SHRP2-R19A-RW-1*** "Bridges for Service Life beyond 100 Years: Innovative Systems, Subsystems and Components" (*Design Guide for Bridges for Service Life* , *Publication S2-R19A-RW-2, Section 3.2.2.10 FRP) 2013.*

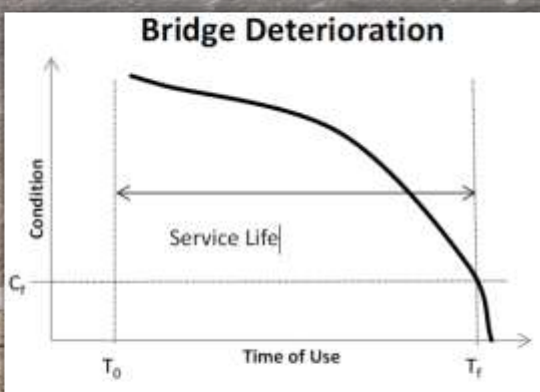


Figure 1.11. An example of bridge deterioration curve.

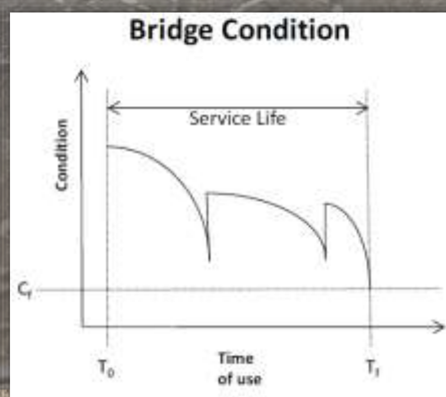
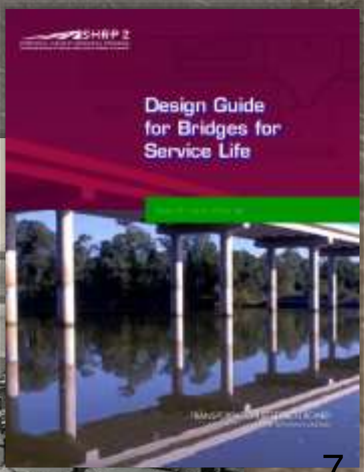
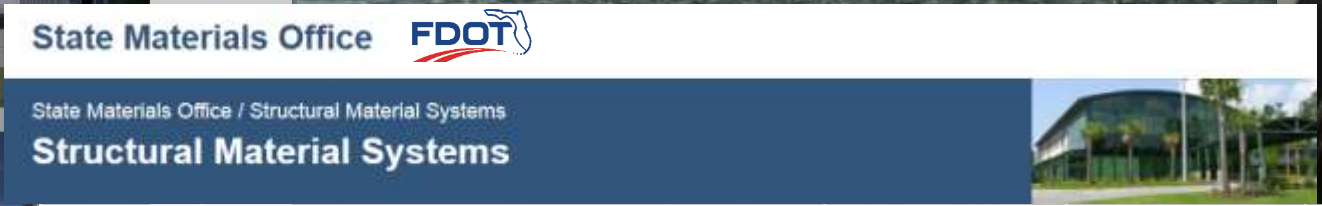


Figure 1.12. Bridge condition life cycle.



Are Composites the Solution?

Service Life Enhancement thru Durability:



6/30/2018	Performance Evaluation of GFRP Reinforcing Bars Embedded in Concrete Under Aggressive Environments	R. Kampmann	Florida State University	BDV30 977-18
3/31/2018	Degradation Mechanisms and Service Life Estimation of FRP Concrete Reinforcements	A. El Safty	University of North Florida	BDV34 977-05
4/16/2014	Investigation of Carbon Fiber Composite Cables (CFCC) in Prestressed Concrete Piles	M. Roddenberry, P. Mtenga	Florida State University	BDK83 977-17
11/30/1998	Studies on Carbon FRP (CFRP) Prestressed Concrete Bridge Columns and Piles in Marine Environment	M Arockiasamy	Florida Atlantic University	B-9076
8/1/1995	Durability of CFRP Pretensioned Piles in Marine Environment Volume II	R. Sen	University of South Florida	0510642

Are Composites the Solution?

Alternate Solutions: Stainless Steel?

Structures Research Center

SRC Home   Active Research

FDOT

State Materials Office   FDOT

State Materials Office / Structural Material Systems

Structural Material Systems

UNIVERSITY OF MIAMI

2018	Type II Prestressed Beams with HSSS	2205 alloy, 0.6" strands, 250 ksi	Florida State University	
2018	Flexural Testing of HSSS Prestressed PIles & Epoxy Dowelled Splices	Potter	FDOT SRC	BDV
2014	VTRC Stainless Steel Piles		Georgia Tech	BDK
	xx			
2013	xx	G. Mullins	University of South Florida	

Are Composites the Solution?

Mitigating Risk of New Technology:

- Acquisition Cost;
- New Material Systems;
- Limited suppliers/competition;
- Unfamiliar design criteria;
- Unfamiliar construction practices.

FDOT

INVITATION TO INNOVATION

AASHTO Innovation Initiative

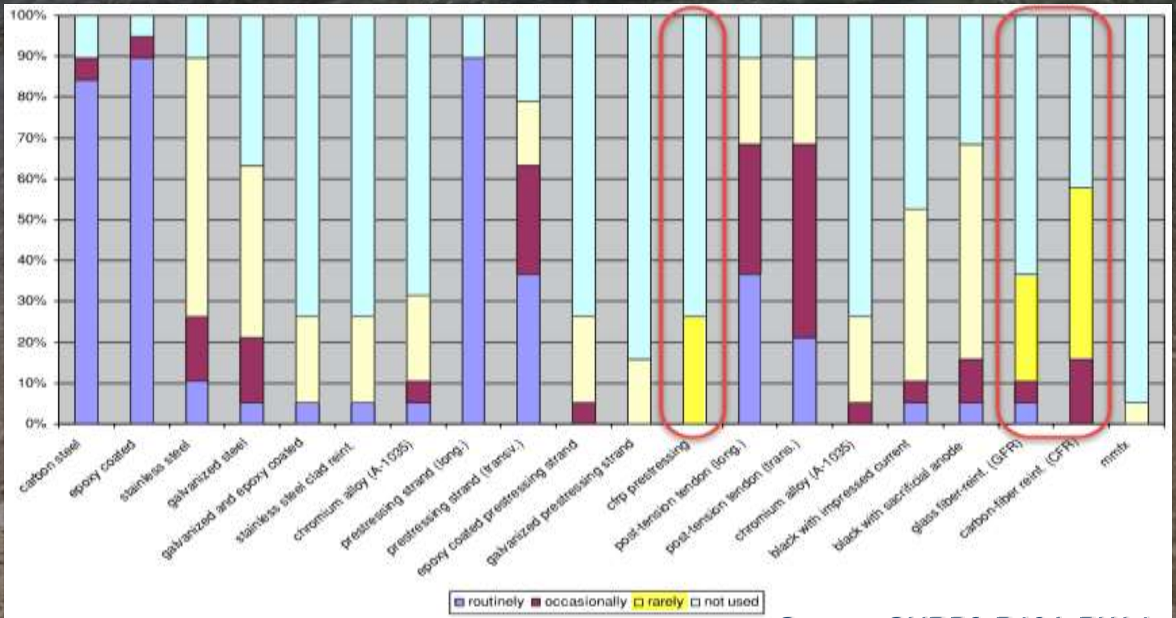


Figure 3.6. Survey results for type of reinforcement used.

Source: SHRP2-R19A-RW-1

History of Seawall Development in Florida

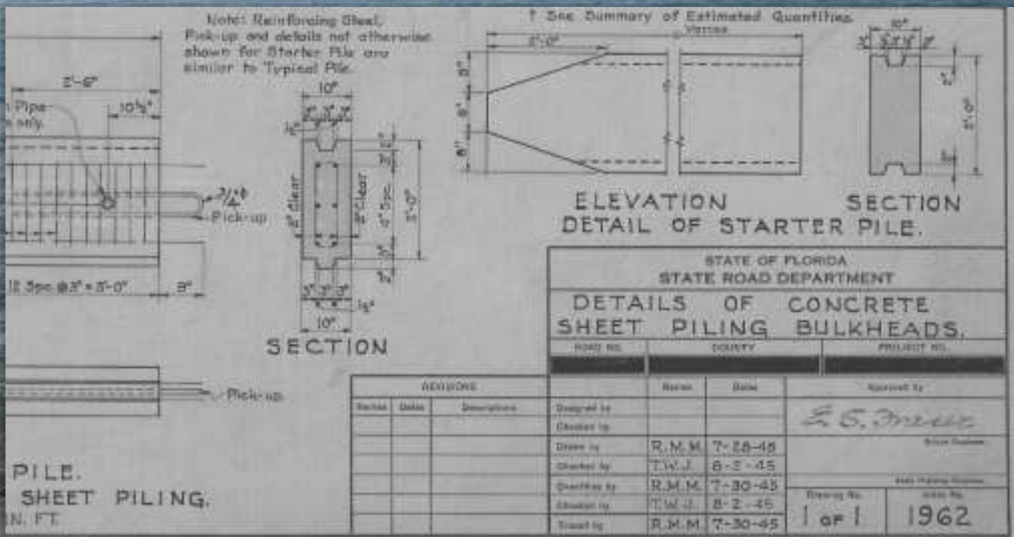
for the Built-Environment...

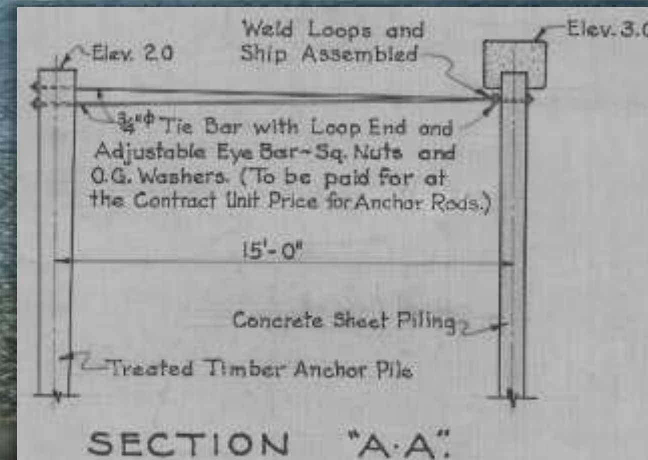
Davis Islands, PCA Concrete Piles Pub., pg.70 (1951)

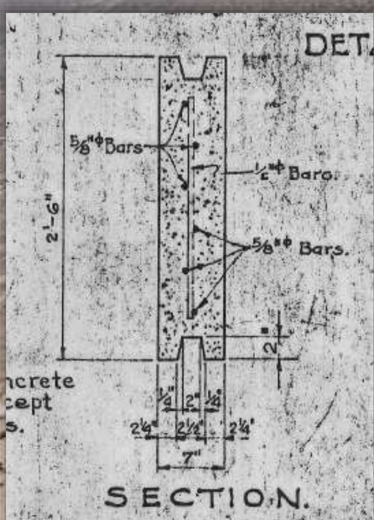
(Photographs courtesy of the Burgert Bros.)

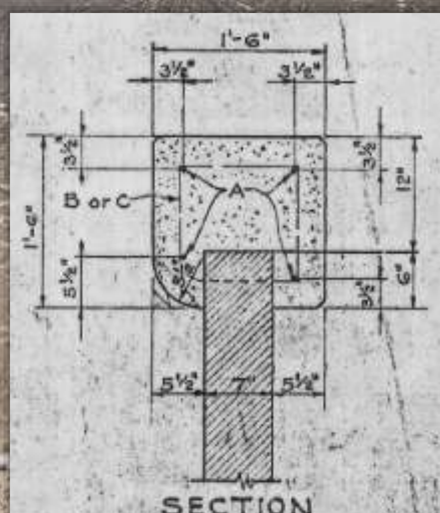
History of Seawall Development in Florida

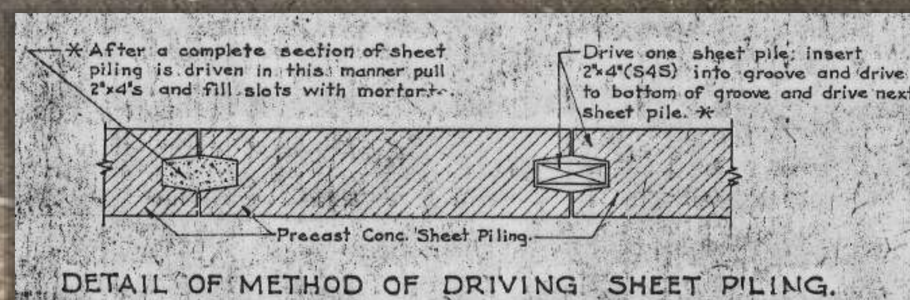
Reinforced Concrete: since 1920's











Images from 1945 (Index 1962) & 1946 (Index 2039) Standards. Florida State Road Dept. (FDOT)

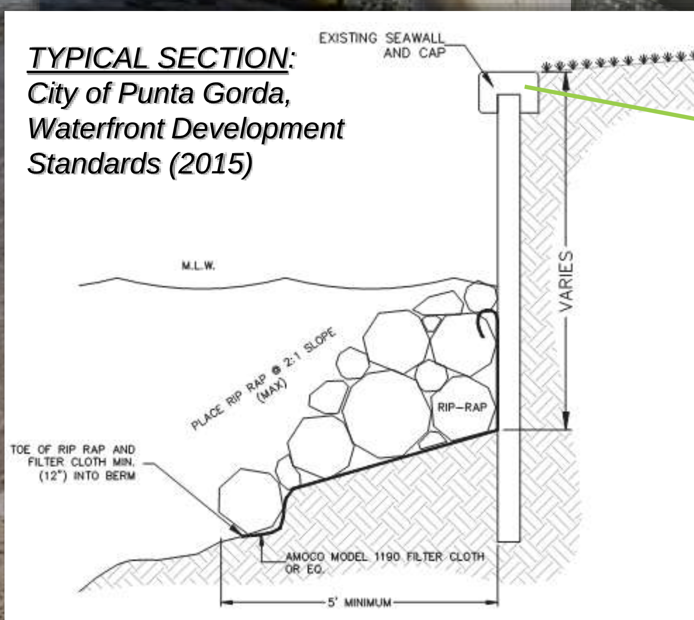
History of Seawall Development in Florida

Reinforced Concrete: since 1920’s

Bulkhead toe failure

Bulkhead wall panel failure

Bulkhead tie-back failure



***TYPICAL SECTION*:** *City of Punta Gorda, Waterfront Development Standards (2015)*

Photos: Courtesy City of Punta Gorda, Waterfront Property Owners Manual (2010)

History of Seawall Development in Florida

Prestressed Concrete: since mid-1950's *…better*


Courtney Campbell Causeway, FDOT (2011)


Courtney Campbell Causeway, Tampa Bay, FDOT (2011)


Courtney Campbell Causeway, Tampa Bay, FDOT (2011)


Replace corroded RC seawalls. Tampa Bay, FDOT (2011)

History of Seawall Development in Florida

FDOT Standards: 1938 - 1978…

Year	Index	Description
1938	1442	Standard H-Pile Bent ~ 24'-0" Roadway. Treated Timber Bulkheads
1939	1501	Bulkhead Details [Timber] for Widening Existing Bridges on the Tamiami Trail
1942	1786	Details of Bulkheads and Splash Walls [Timber]
1942	1819	Standard Treated Timber Bulkhead-26 Ft. Roadway
1945	1943	Standard Treated Timber Bulkhead
1945	1962	Details of Concrete Sheet Piling Bulkheads
1946	2039	Details of Concrete Sheet Piling Bulkhead
1948	2260	Details of Concrete Sheet Piling Bulkhead
1950	2467	Precast Concrete Sheet Piling
1951	2615	Concrete Bulkheads-15' Span-28' Roadway-2~3'-1" Sidewalks
1951	2686	Details of Special Sections Concrete Sheet Piling Bulkhead
1952	2692	Precast Cantilever Bulkhead
1953	3040	Concrete Sheet Piling Bulkheads
1954	3204	Concrete Bulkheads with Composite Master Piles and Precast Slabs
1954	3235	Concrete Sheet Piling Bulkhead
1954	3388	Precast Concrete Sheet Piling
1957	3904	Precast Concrete Sheet Piling
1957	3950	Precast Concrete Sheet Piling (10" x 30")
1957	4051	Concrete Sheet Piling Bulkhead Details
1957	4057	Precast Concrete Sheet Piling (8" x 30") [Prestressed]
1957	4059	Precast Concrete Sheet Piling (12"x30") [Prestressed]
1959	4513	Precast Concrete Sheet Piling (7" x 30") [Prestressed]
1961	7284	Precast Concrete Sheet Piling (8" X 30") [Prestressed]
1970	3950rev	Precast Concrete Sheet Piling (10" x 30") [Prestressed]

History of Seawall Development in Florida

FDOT Standards: 1978 – 2018 *(Prestressed)*

Year	Index	Description
1978		...continued
1989	610	10" Concrete Sheet Piles
1996	610	10" Concrete Sheet Piles
1998	610	Precast Concrete Sheet Piles -Type "A" (Prestressed)
1998	611	Precast Concrete Sheet Piles - Variable Angle Corner Pile
1998	612	Precast Concrete Sheet Piles - Right Angle Corner Pile
1998	S-613	Precast Concrete Sheet Piles
2000	S-650	Precast Concrete Sheet Pile [LRFD]
2000	660	Precast Concrete Sheet Pile Type "A' – 10 Inch Thick
2000	670	Precast Concrete Sheet Pile Type "A" – 12 Inch Thick
2000	680	Precast Concrete Sheet Pile Type "B" – Variable Angle Corner Pile
2000	690	Precast Concrete Sheet Pile Type "C" – Right Angle Corner Pile
2003	S-400	Precast Concrete Sheet Pile
2003	410	Precast Concrete Sheet Pile Type "A' – 10 Inch Thick
2003	412	Precast Concrete Sheet Pile Type "A" – 12 Inch Thick
2003	430	Precast Concrete Sheet Pile Type "B" – Variable Angle Corner Pile
2003	440	Precast Concrete Sheet Pile Type "C" – Right Angle Corner Pile
2006	20400	Precast Concrete Sheet Pile Wall
2008	6040	Precast Concrete Sheet Pile Wall
2014	D22440	Precast Concrete CFRP/GFRP Sheet Piles
2017	22440	Precast Concrete CFRP/GFRP & HSSS/GFRP Sheet Pile Wall
2018	455-400	Precast Concrete Sheet Pile Walls (Conventional)
2018	455-440	Precast Concrete Sheet Pile Walls (CFRP/GFRP & HSSS/GFRP)

History of Seawall Development in Florida

CFRP Prestressing, since 2014 *…best ?*

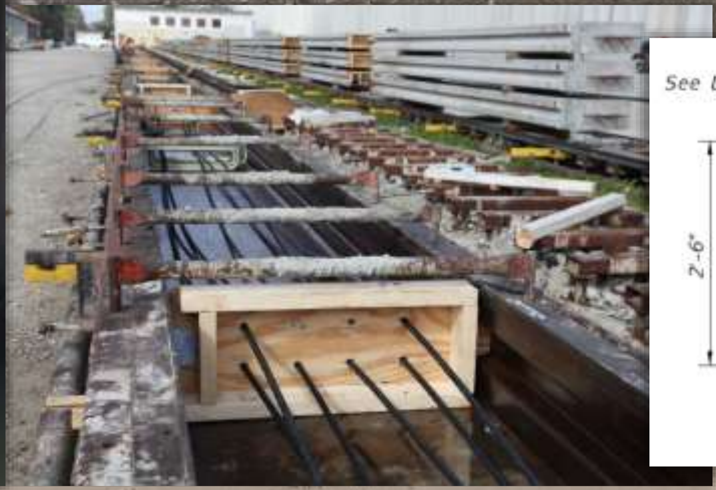
i. Design criteria for prestressing – ***Fiber Reinforced Polymer Guidelines*** ***(FRPG)*** – *Chapter 3*;

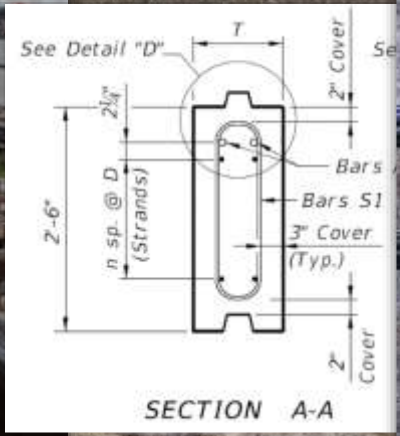
ii. ***Developmental*** Index D22440 *(Nov. 2014)*

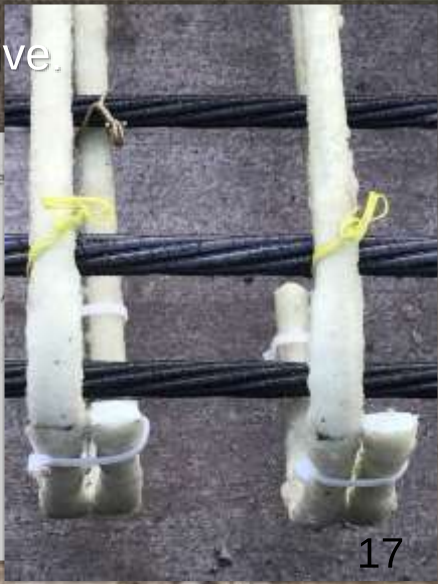
- (Halls River Bridge demonstration project);

iii. FDOT ***FY2017-18 Design Standards*** *(Nov. 2016)*

- Index 22440 series;
- CFRP prestressing strands & GFRP stirrups;
- Stainless Steel prestressed/reinforced alternative.







"Current" Seawall Deployment by FDOT

FDOT Standards 2018:

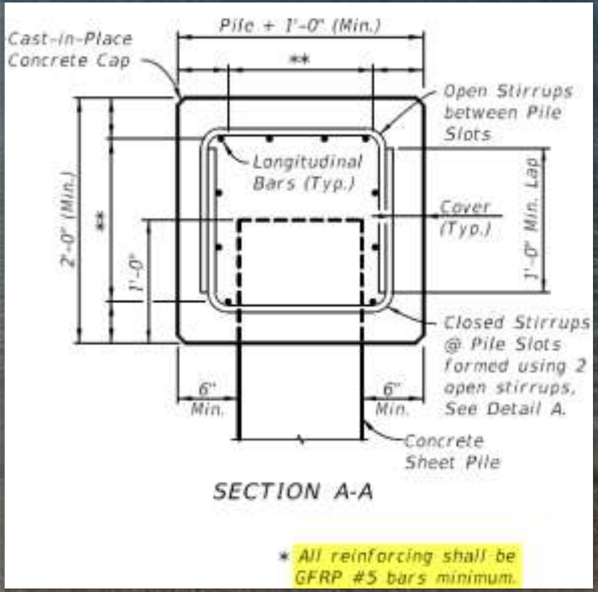
i. C-I-P **GFRP-RC** Cap details for concrete sheet pile walls;

ii. FDOT ***Specifications*** *415 & 932* ***(GFRP rebar)***

iii. Approved Producers List requirements via ***Materials Manual*** *– Section 12.1 (Jan 2015);*

iv. Design criteria for rebar – ***Fiber Reinforced Polymer Guidelines (FRPG)*** *– Chapter 2;*

v. Standard detailing - ***Structures Detailing Manual*** ***(SDM)*** *- Chapter 19.5.1 and special GFRP Instructions* ***IDDS****-D22440* *(Jan 2015).*



“Future” of Seawall Development in Florida

Future systems, …2020? *…bestest!!*

Developing design criteria for:

i. ***Glass-FRP*** prestressing

ii. ***Basalt-FRP*** reinforcing

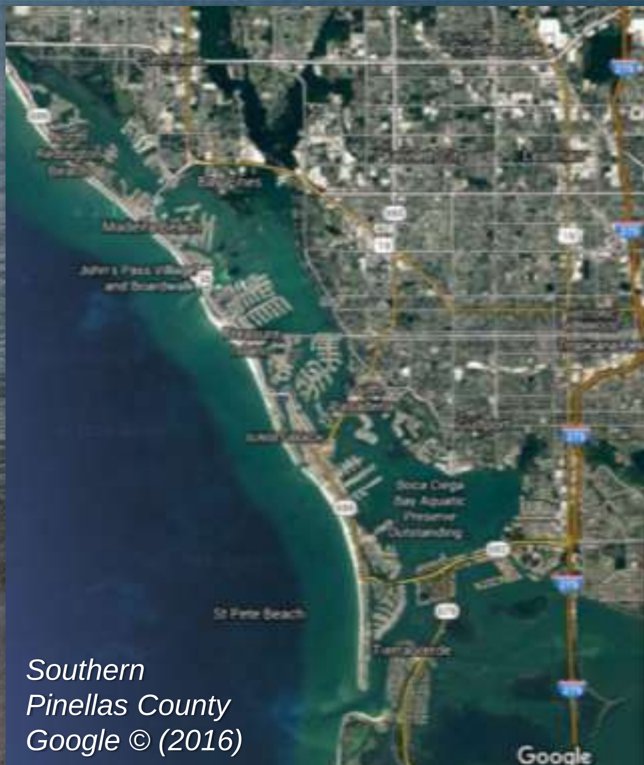
FHWA's ***I****nnovations* ***D****eserving of* ***E****xploratory* ***A****nalysis* ***(IDEA)***

- GFRP Prestressing - ***MILDGLASS*** *(University of Miami);*

FHWA's ***S****tate* ***T****ransportation* ***I****nnovation* ***C****ouncils* ***(STIC)*** Incentive Program

- BFRP Reinforcing Standards Development -***MACTBr*** *(FDOT)*

Quantifying the Built-Shoreline Legacy?



Southern Pinellas County Google © (2016)

Fort Lauderdale Google © (2016)


Scientists Wary of Fort Lauderdale's Proposed Seawall Plan

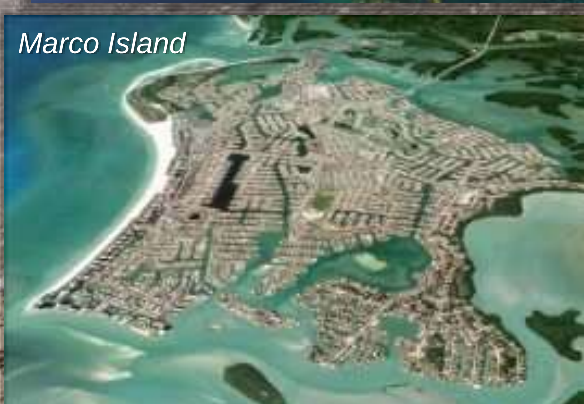
BY JESS SWANSON


Sunshine Skyway Rest Area

Original story:

Fort Lauderdale has over 200 miles of seawalls. During high tides and storms, seawalls protect properties from coastal flooding. Currently, a city ordinance dictates that seawalls be no higher than five and a half feet. But during King Tides, the really high tides in September, the seawalls are not cutting it. In coastal communities like Las Olas Isles, water is already washing over them and nearly flooding homes. Experts fear this flooding will get worse as sea level is predicted to rise.

It is estimated to cost a property owner anywhere from $10,000 to $125,000 to raise an existing seawall or completely replace a 100-foot seawall. With four miles of public seawalls, it can cost the city as much as $26 million to replace its seawalls. Slap worries about spending


Marco Island

Quantifying the Built-Shoreline Legacy

RESEARCH COMMUNICATIONS RESEARCH COMMUNICATIONS

Engineering away our natural defenses: an analysis of shoreline hardening in the US

Rachel K Gittman[1*], F Joel Fodrie[1], Alyssa M Popowich[1], Danielle A Keller[1], John F Bruno[1], Carolyn A Currin[4], Charles H Peterson[1], and Michael F Piehler[1]

Rapid population growth and coastal development are primary drivers of marine habitat degradation. Although shoreline hardening or armoring (the addition of concrete structures such as seawalls, jetties, and groins), a byproduct of development, can accelerate erosion and loss of beaches and tidal wetlands, it is a common practice globally. Here, we provide the first estimate of shoreline hardening along US Pacific, Atlantic, and Gulf of Mexico coasts and predict where future armoring may result in tidal wetland loss if coastal management practices remain unchanged. Our analysis indicates that 22 842 km of continental US shoreline – approximately 14% of the total US coastline – has been armored. We also consider how socioeconomic and physical factors relate to the pervasiveness of shoreline armoring and show that housing density, gross domestic product, storms, and wave height are positively correlated with hardening. Over 50% of South Atlantic and Gulf of Mexico coasts are fringed with tidal wetlands that could be threatened by future hardening, based on projected population growth, storm frequency, and an absence of coastal development restrictions.

Front Ecol Environ 2015; 13(6): 301–307, doi:10.1890/150065

Although coastal regions constitute less than 4% of the Earth's land area, coastal habitats (eg b… tidal wetlands; Figure 1) rank among the mo… natural resources globally (MA 2005). Over o… the human population lives within 100 k… … also provide less physically complex habitat as compared

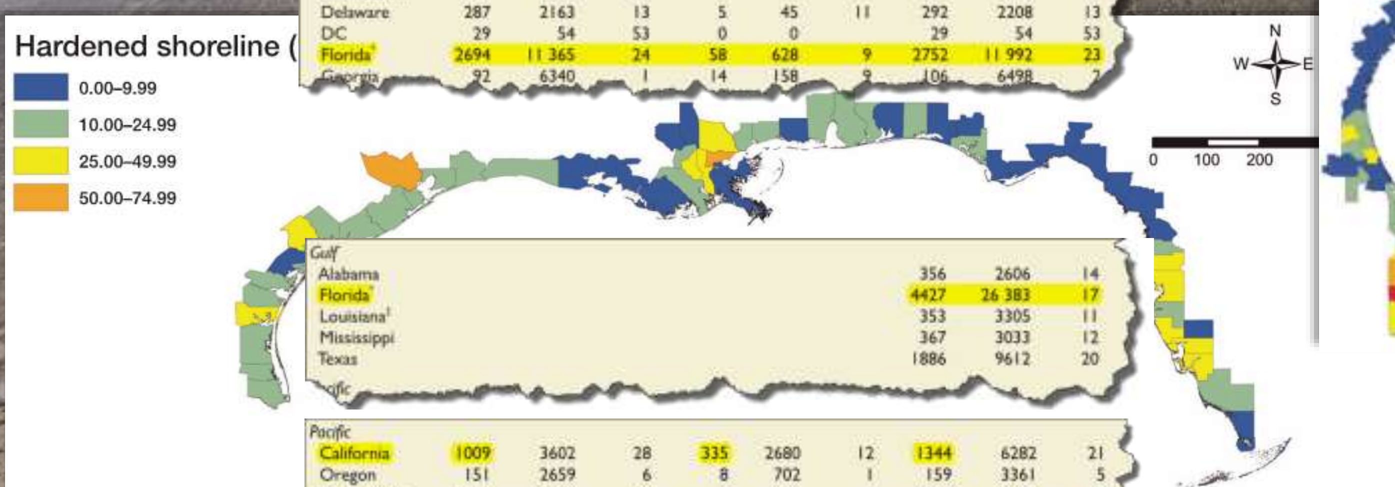
WebTable 2. Summary of shoreline hardening estimate for continental US coasts

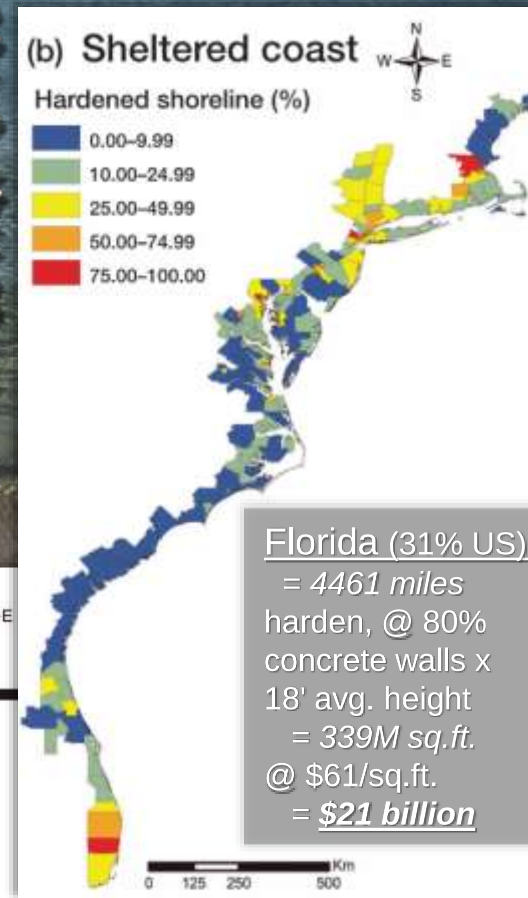
Region	Total shoreline (km)	Hardened shoreline (km)	Hardened (%)
Atlantic	99 494	12 923	13
Sheltered	93 848	12 425	13
Open	5646	498	9
Gulf of Mexico[†]	44 939	7390	16
Pacific	15 735	2529	16
Sheltered	12 026	2182	18
Open	3709	348	9
Total	160 168	22 842	14

Notes: [†]The Gulf of Mexico shoreline could not be divided into "Open"

WebTable 3. Shoreline hardening and population statistics by state

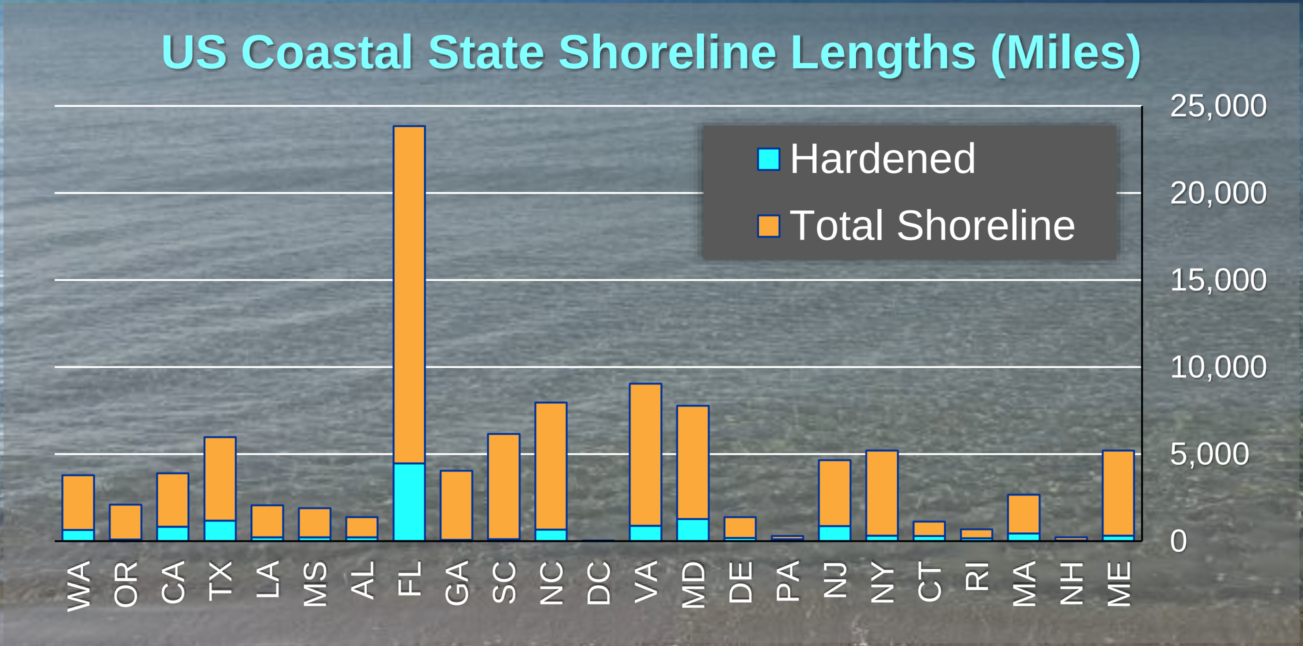
	Hard sheltered shore (km)	Sheltered shore (km)	Hard sheltered shore (%)	Hard open shore (km)	Open shore (km)	Hard open shore (km)	Hard shore (km)	Total shore (km)	Hard shore (%)
Atlantic									
Connecticut	477	1907	25	0	0		477	1907	25
Delaware	287	2163	13	5	45	11	292	2208	13
DC	29	54	53	0	0		29	54	53
Florida[†]	2694	11 365	24	58	628	9	2752	11 992	23
Georgia	92	6340	1	14	158	9	106	6498	7
Gulf									
Alabama							356	2606	14
Florida[†]							4427	26 383	17
Louisiana[†]							353	3305	11
Mississippi							367	3033	12
Texas							1886	9612	20
Pacific									
California	1009	3602	28	335	2680	12	1344	6282	21
Oregon	151	2659	6	8	702	1	159	3361	5
Washington	1022	5765	18	5	327	1	1027	6092	17

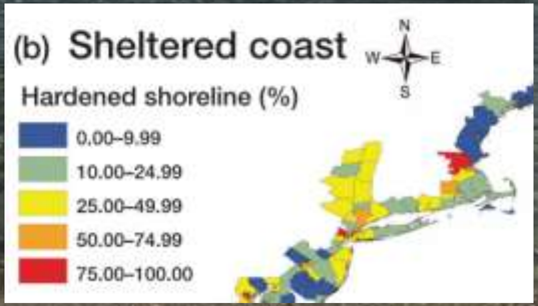




Florida (31% US):
= *4461 miles*
harden, @ 80%
concrete walls x
18' avg. height
= *339M sq.ft.*
@ $61/sq.ft.
= ***$21 billion***

Quantifying the Built-Shoreline Legacy

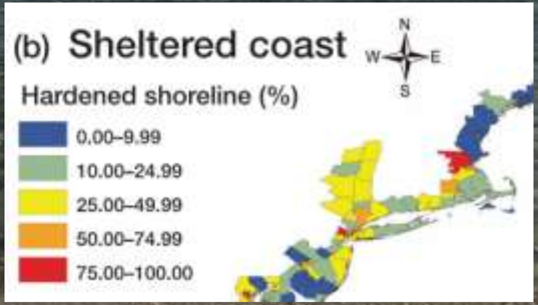




Quantifying the Built-Shoreline Legacy

Florida Municipal Examples:

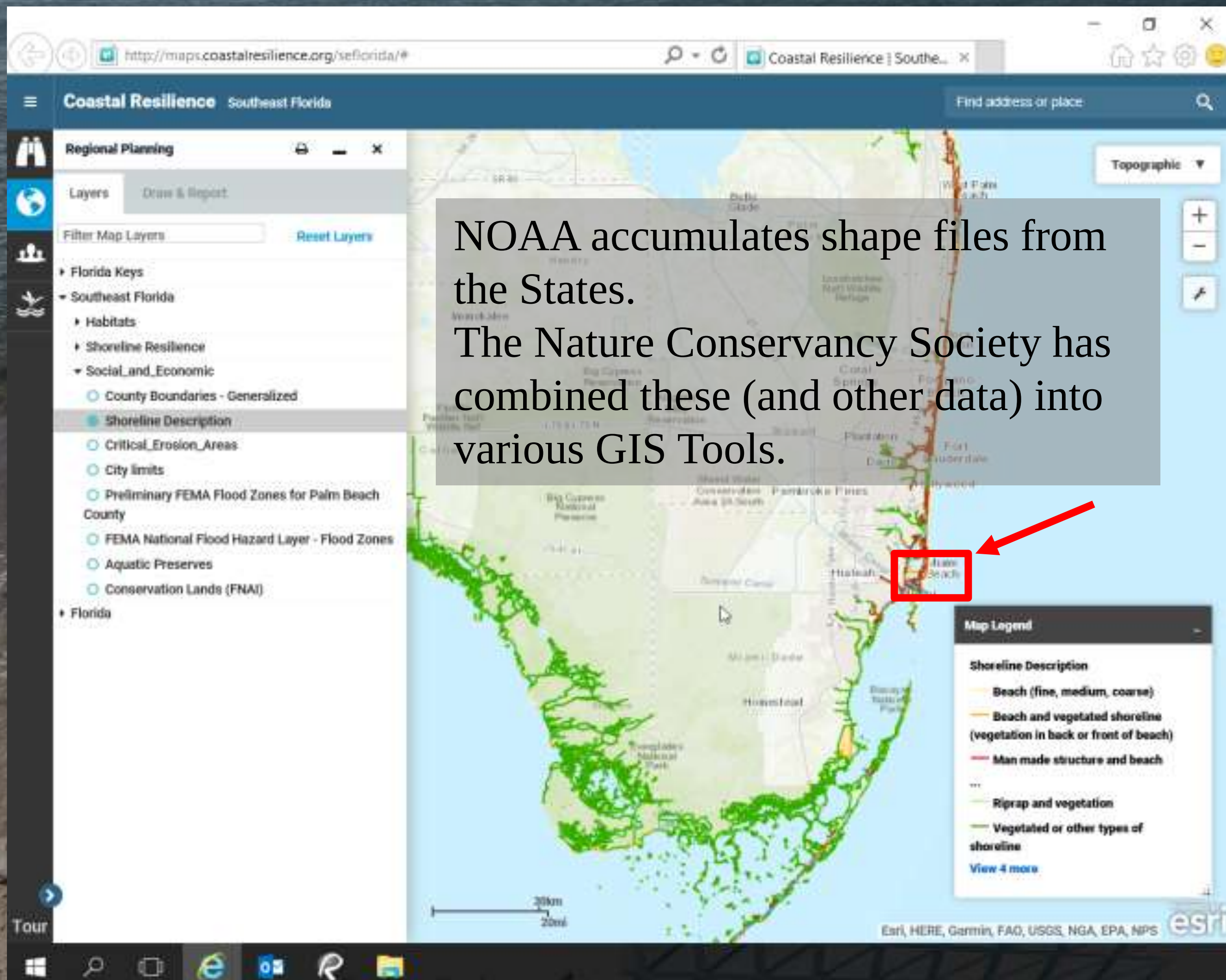
- City of Miami Beach = 60+ miles
- City of Miami = 11+ miles
- City of Punta Gorda = 124 miles
- Marco Island = 200+ miles
- Fort Lauderdale = 200+ miles
- Tampa Bay Area = ??? (*Davis Islands 11.5 miles*)
- Monroe Co. (2003) = 222 miles residential canals (*rubble or bulkhead*)
- …



Quantifying the Built-Shoreline Legacy

GIS-Mapping Tools:

http://coastalresilience.org/project/coastal-resilience-mapping-tool-overview/

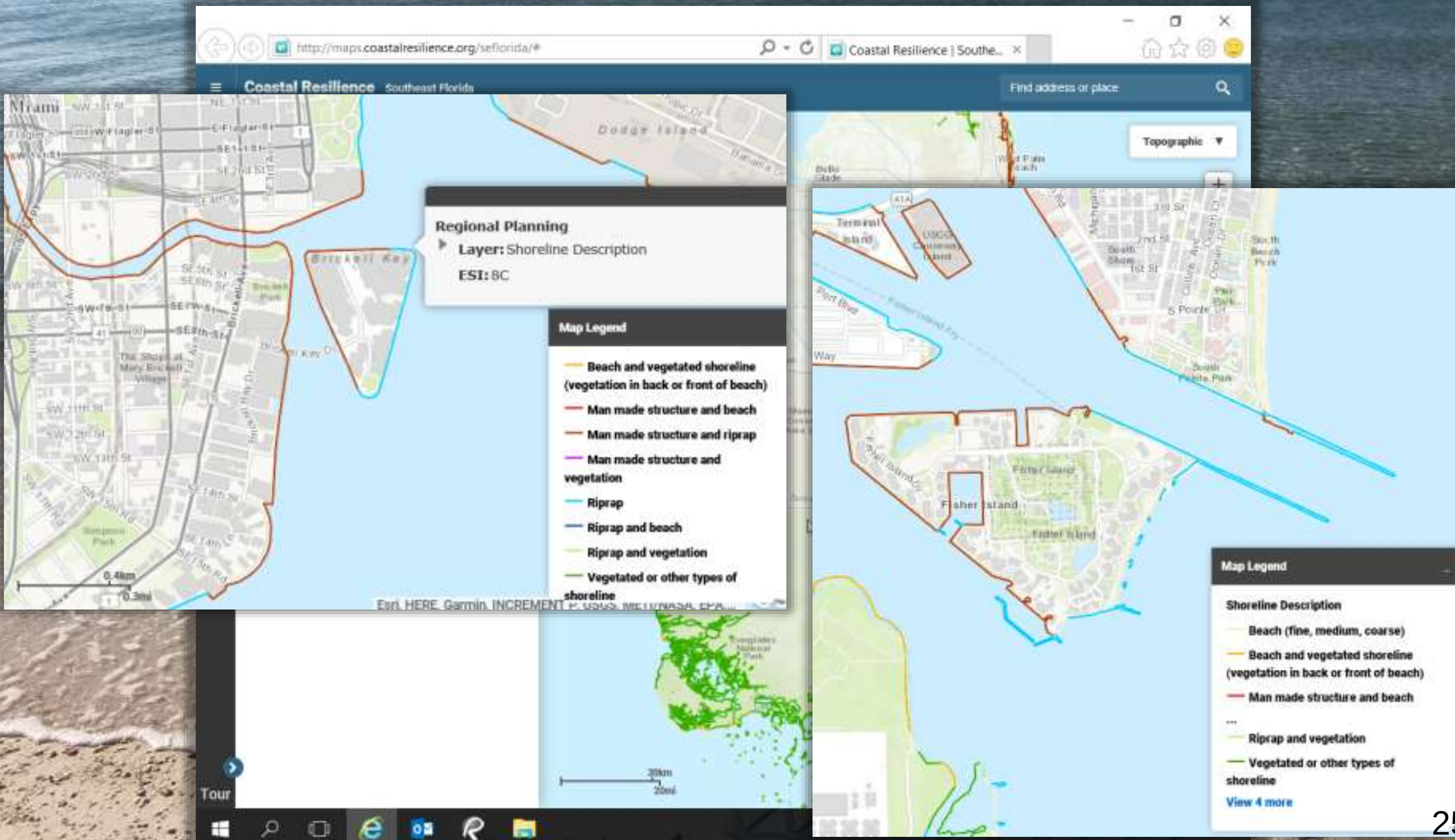


NOAA accumulates shape files from the States.
The Nature Conservancy Society has combined these (and other data) into various GIS Tools.

Quantifying the Built-Shoreline Legacy

GIS-Mapping Tools:

http://coastalresilience.org/project/coastal-resilience-mapping-tool-overview/



Quantifying the Built-Shoreline Legacy

Typical Examples (Miami):

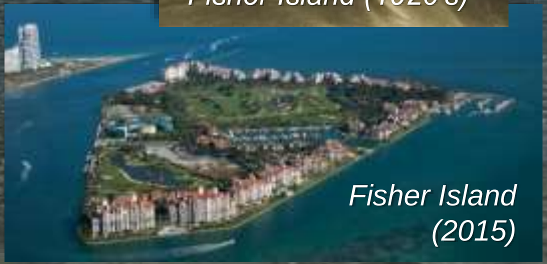
- **Fisher Island** (1919) = 2.4 mile
- **Brickell Key** (1920's-70's) = 1.1 mile
- **Dodge Island** (1950's) = 4.2 miles
- **Key Biscayne** (1950's-70's) = 8 miles


Brickell Key (Burlingame Is.,1928)


Fisher Island (1920's)


Miami, Brickell Bay Dr. (1915)


island marine
Photo - Nov 2011


Fisher Island (2015)

The Mackle Company developed the middle of Key Biscayne in the 1950's


Miami, Brickell Bay Dr. Google © (2015)


Miami, Key Biscayne (2015)


Brickell Key, (Claughton Is., 1972)

Quantifying the Built-Shoreline Legacy

Typical Examples:

- **Marco Island** ("The Platinum Coast", Collier Co. – 1960's) ~ 200 miles

http://www.themacklecompany.com/femjrstorypublic/16-deltona-marcoisland.htm

Under the direction of Bill O'Dowd and Earl Cortright Sr. a high production sea-wall operation - refining the processes developed at Key Biscayne, Pompano and Port Charlotte - was planned. Land based drag lines along with water based dredges would do the major earth moving.

Marco Island progress, 1967.
(Image courtesy of the Mackle Company)

Marco Island 1964, and early residents!
(Images courtesy of the Mackle Company)

Marco Island showroom scale model, 1965.
(Image courtesy of the Mackle Company)

Marco Master Plan, 1969.
(Image courtesy of the Mackle Company)

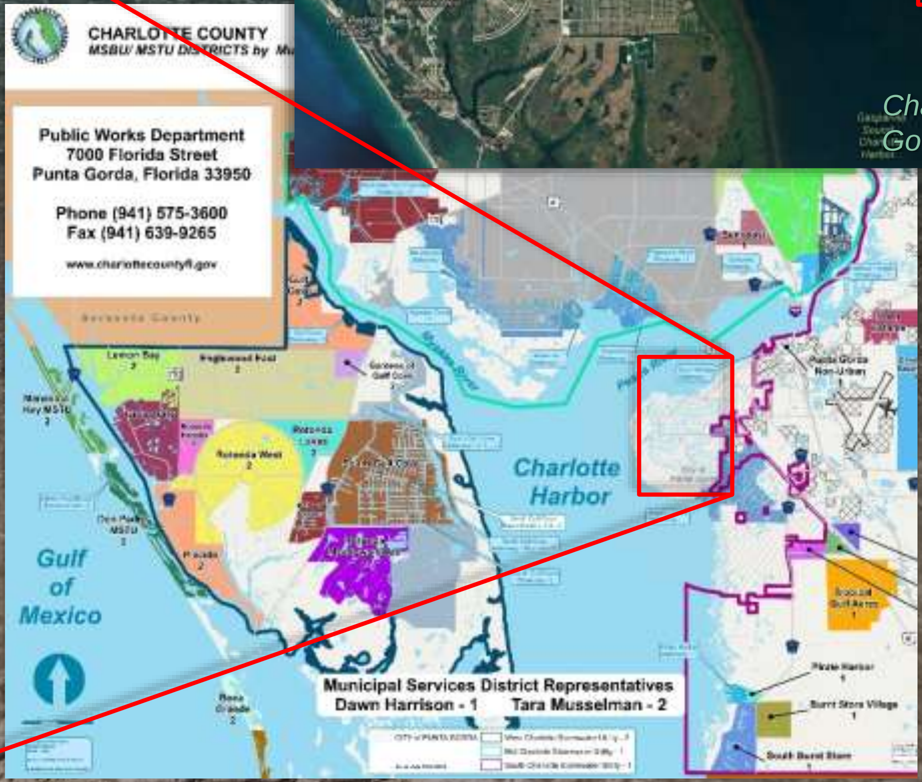
Quantifying the Built-Shoreline Legacy

Typical Examples:

- **City of Punta Gorda** (Punta Gorda Isles 1960's-70's & Burnt Store Isles 1970's-80's) = 124 miles
- **Charlotte Co.** (MSBU-Waterway Districts) = ???

(MMFX article: http://mmfxsteelcorporation.cmail2.com/t/t-l-hydhdjl-klkyuthuu-p/)

Charlotte Harbor
Google © (2016)



Quantifying the Built-Shoreline Legacy

Typical Examples:

Davis Islands (Tampa – 1920's) = 11.5 miles
Davis Shores (St. Augustine 1920's - 60's) = 2 miles


Davis Islands (1924)


Davis Islands, Dredge-and-fill (1926)


Davis Islands, PCA Concrete Piles Pub., pg.70 (1951)


Davis Islands (1925)


Davis Islands, Adalia Ave (1926)


Davis Islands (1926)

North Davis Shores is part of a larger "City Beautiful" project that developer D. P. Davis sought to create on the island in the 1920s soon after the success of Davis Island in Tampa. The City Beautiful concept included stately plazas, embellished boulevards, waterside promenades, prominent public statues, fountains and memorials.

If it had not been for the Florida real estate bust during the Great Depression and the disappearance of the developer, the project would have outshone his other examples of

Quantifying the Built-Shoreline Legacy

Typical Examples:

Pinellas county (Tampa Bay, 1910's – 1960's)

"The 'finger island' frenzy - the dredging of islands just wide enough for a cul-de-sac road and houses on either side—reached its heyday between the mid-1940s and 1960s. During this period, developers throughout the Tampa Bay region reaped the riches of a second land boom" ([1] James Anthony Schnur, 2015)

Dredging operations transformed Boca Ciega Bay along Redington Beach and Madeira Beach. (Image courtesy of Archives and Library, Heritage Village) [1]

Dredging operations transformed Paradise Island and the Yacht Club Estates along the Treasure Island Causeway during the 1950s. (Image courtesy of Archives and Library, Heritage Village) [1]

Quantifying the Built-Shoreline Legacy

How Much $$$

Replacement liability estimate:

	Hard Shore (km)	Total Shore (km)	Hard Shore (%)
Total (km)	**7,177**	**38,372**	**19%**
Total (miles)	**4,460**	**23,843**	
Open Shoreline (km)	58	626	9%
Open Shoreline (miles)	36	389	
Sheltered (km)	7,119	37,746	19%
Sheltered (miles)	4,424	23,454	

Using FDOT average unit rate of $61/sq.ft *(2009-2016)*.
Assuming 80% of the hardended shoreline is concrete sheet pile,
and the exposed wall height averages 6 ft. and buried length is 12 ft.
The total replacement cost in present day dollars is approximately **$21 Billion**

New Challenges

SLR, Extreme Weather, Sustainability,
Increased Durability Expectations

New Challenges

SLR, Extreme Weather, Sustainability, Increased Durability Expectations

(a) Hurricane Damage along A1A (*2008)*
(b) Hurricane Ivan damage in Escambia Bay (*2004)*

New Challenges

SLR, Extreme Weather, Sustainability, Increased Durability Expectations

- **(a) Hurricane Mathew damage along A1A Flagler Beach, *(2016)***
- **(b) Pensacola Pier most days of the year …**
- **(c) Pensacola Pier during Hurricane Nate *(2017)***

New Challenges

SLR, Extreme Weather, Sustainability, Increased Durability Expectations

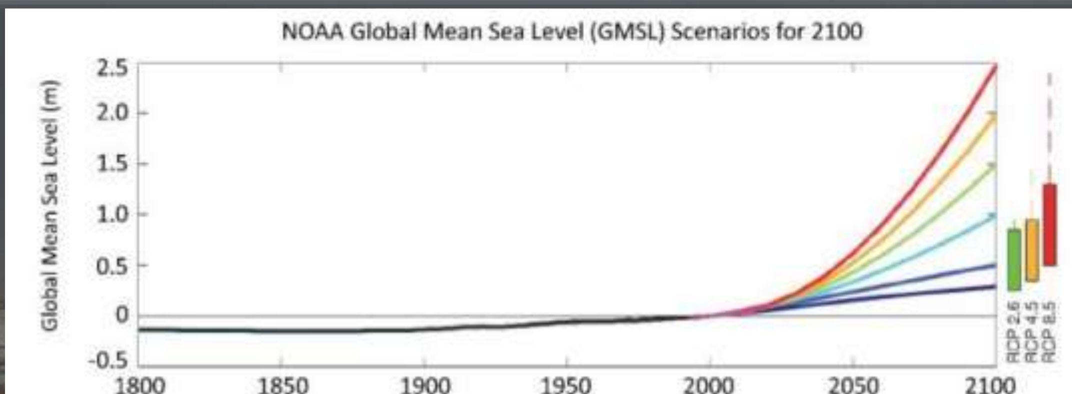


Figure 8. This study's six representative GMSL rise scenarios for 2100 (6 colored lines) relative to historical geological, tide gauge and satellite altimeter GMSL reconstructions from 1800–2015 (black and magenta lines; as in Figure 3a) and central 90% conditional probability ranges (colored boxes) of RCP-based GMSL projections of recent studies (Church et al., 2013a; Kopp et al., 2014; 2016a; Slangen et al., 2014; Grinsted et al., 2015; Mengel et al., 2016). These central 90% probability ranges are augmented (dashed lines) by the difference between the median Antarctic contribution of Kopp et al. (2014) probabilistic GMSL/RSL study and the median Antarctic projections of DeConto and Pollard (2016), which have not yet been incorporated into a probabilistic assessment of future GMSL. (A labeling error in the x-axis was corrected on January 30, 2017).

AIA Florida

Press Release

For Immediate Release
April 19, 2016

Contact: Candace Munz

AIA Florida Publishes Position on Future Sea Level Rise

(a) NOAA projections under GMSL rise scenarios *(2017)*;
(b) American Institute of Architects Florida: Position Statement on Future Sea Level Rise *(2016)*;
(c) Hurricane Sandy damage along A1A in Fort Lauderdale *(Photo: Susan Stocker, Sun Sentinel, 2012)*.

New Challenges

SLR, Extreme Weather, Sustainability, Increased Durability Expectations

(a) Miami Beach seawall bulkhead and road raising project. *(Photo / Bruce Mowry, city engineer (2017);*
(b) Brickell Ave under water during Hurricane Irma *(2017)*
(c) Inevitable corrosion of prestressing steel in concrete structures in marine environments;

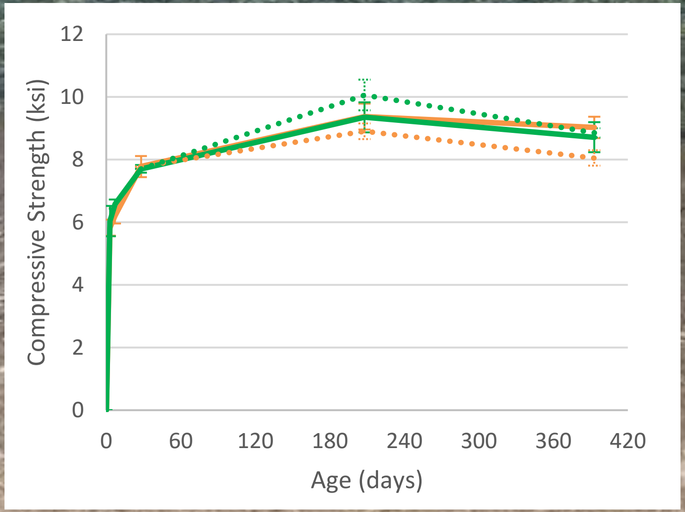
New Solutions

SEACON, GFRP-PC, BFRP (STIC)

New Solutions

SEACON…

Sustainable concrete using seawater, salt-contaminated aggregates, and non-corrosive reinforcement



Infravation
An Infrastructure Innovation Programme

U.S.Department of Transportation
Federal Highway Administration

New Solutions

SEACON CONSORTIUM

Partners:

- **University of Miami (UM)**
- **ATP srl (ATP)**
- **Politecnico di Milano (POLIMI)**
- **Owens Corning (OC)**
- **Buzzi Unicem (BUZZI)**
- **Acciaierie Valbruna (AV)**

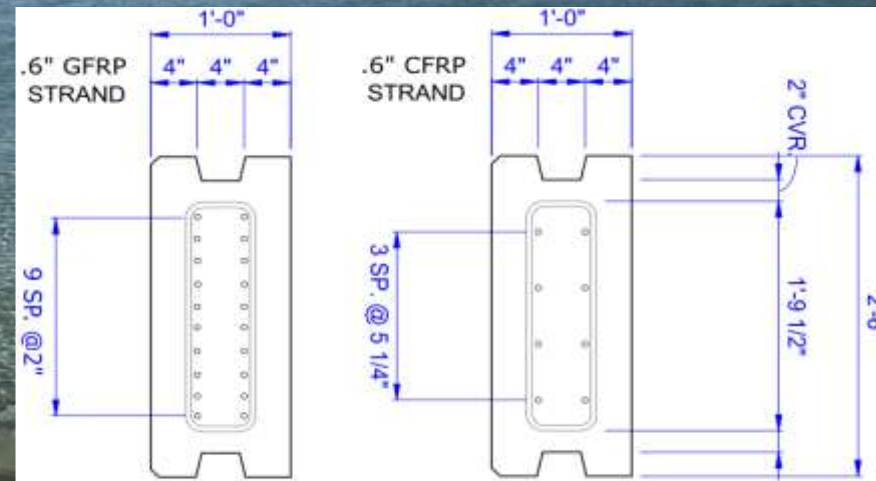
Collaborators:

- **Florida Department of Transportation (FDOT)**
- **Pavimental (PV)**
- **Titan America (TT)**
- **ITC-CNR (ITC), since August 2017**

New Solutions

IDEA Project - *MILDGLASS*

(a) & (b) CFRP strand failed during tensioning; (c) cracking following strands release.



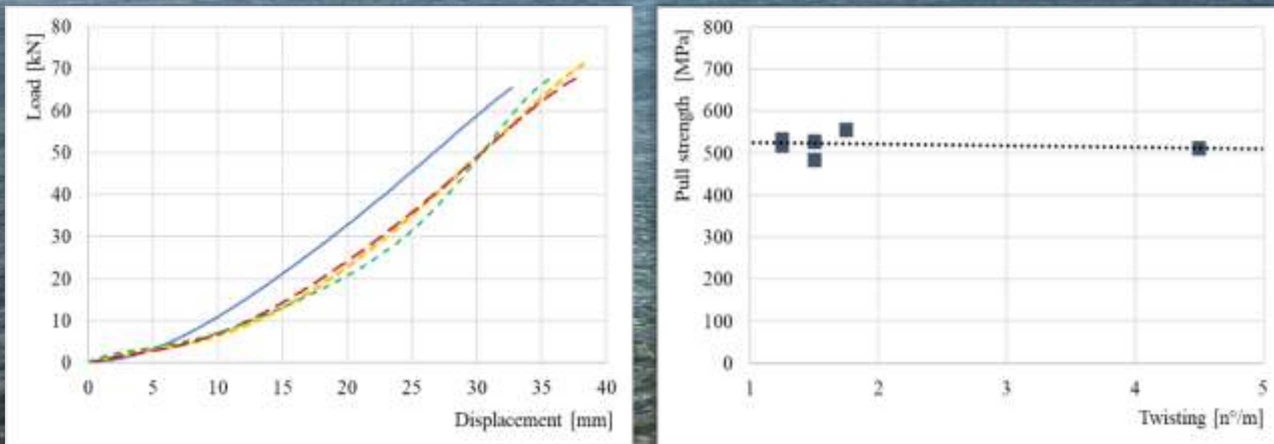
(a) GFRP-PC sheet pile concept (b) CFRP-PC sheet pile design for Halls River Bridge

(a) GFRP strand prototype cross section; (b) compared to a CFRP alternative.

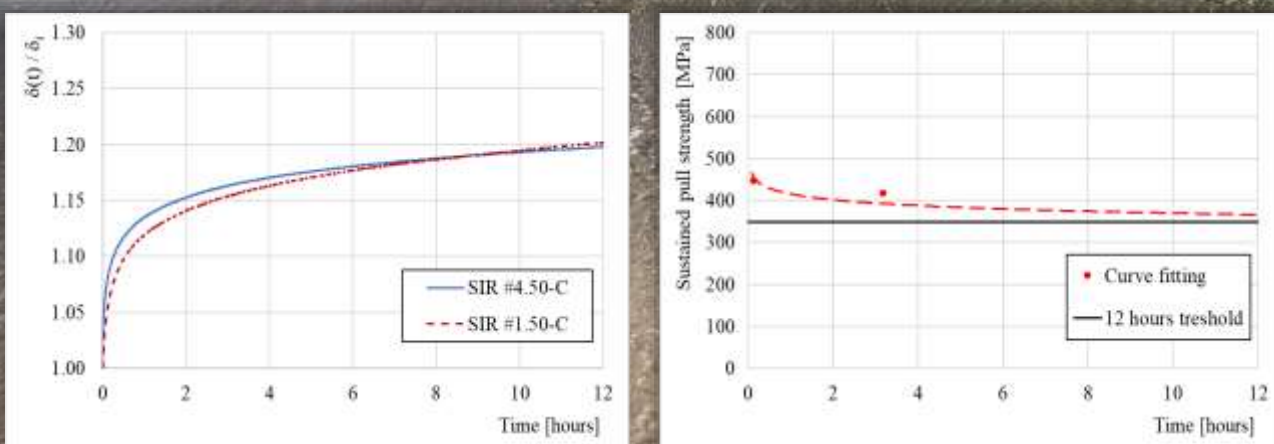
(a) & (b) Tensioning apparatus for CFRP; versus (c) standard steel HSCS chucks, for GFRP.

New Solutions

IDEA Project - *MILDGLASS*



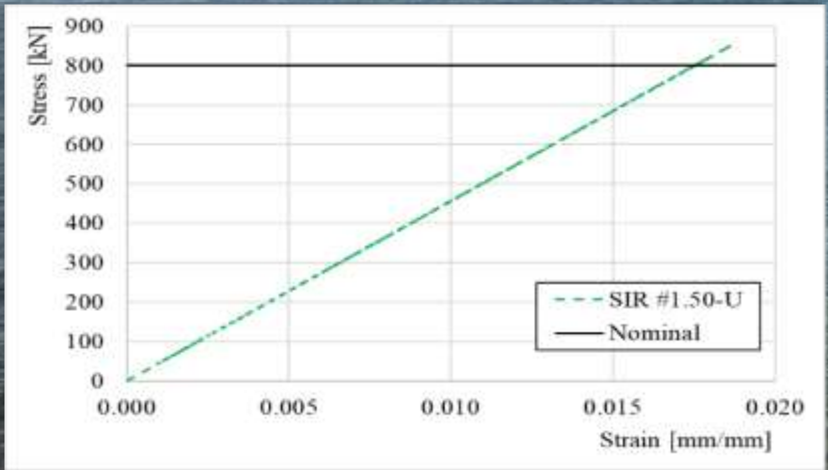
Pull test load-displacement diagrams (a), and pull strength at varying twist per meter (b).

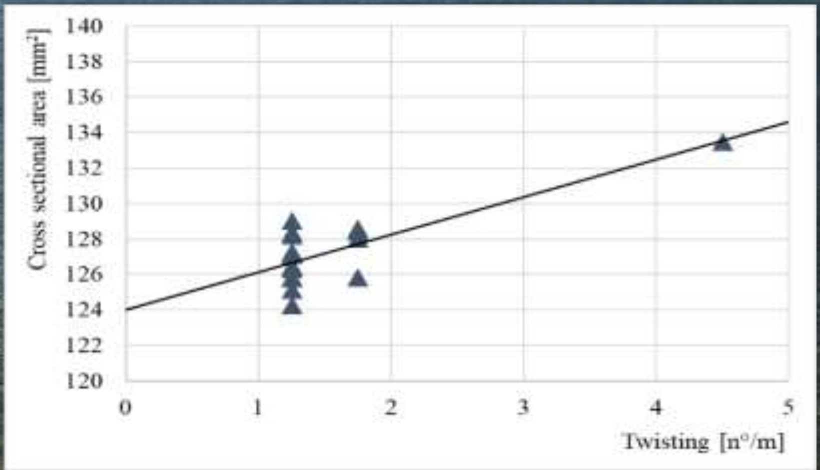


Creep displacement over initial value (a), and creep rupture logarithmic regression (b).

New Solutions

IDEA Project - *MILDGLASS*





(a) Tensile test stress-strain diagram, (b) cross-sectional area at varying twist per meter.

(a) GFRP strand ready for pull test;
(b) After tensile pull test at showing wedge grip;
(c) and surface beneath wedge grip .

New Solutions

IDEA Project - *MILDGLASS*

Table 1 – GFRP-PC sheet pile design compared to HSCS, HSSS, and CFRP alternatives.

Material system	*Strand cost*	$f_{f,pull}$	P_{pull}	Δ_e	P_i	$\sigma_{c,i}$	Δ_{75y}	P_{75y}	$\sigma_{c,75y}$
	$/m	-	kN	%	kN	MPa	%	kN	MPa
HSCS (1x7, 15.2mm)	26	0.85 f_{fu}^*	1 673	9.5%	1,513	10.9	29.0%	1,187	8.5
HSSS (1x7, 12.7mm)	105	0.85 f_{fu}^*	1 078	6.7%	1,005	7.2	22.5%	836	6.0
CFRP (1x7, 15.2mm)	100	0.65 f_{fu}^*	1 264	7.4%	1,170	8.4	23.9%	961	6.9
GFRP (1x7, 15.2mm)	68	0.44 f_{fu}^*	804	5.5%	760	5.5	14.7%	686	4.9

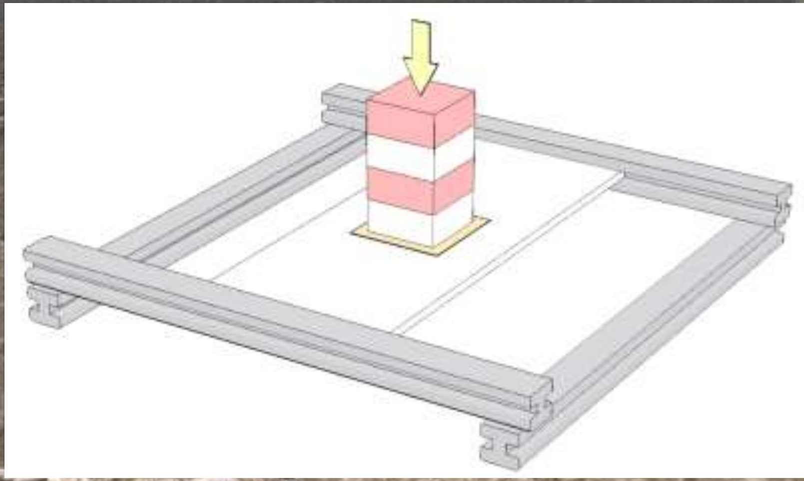
PROTOTYPE INVESTIGATION CONCLUSIONS

The relatively low modulus of elasticity of GFRP, resulted in losses estimated to be half the value associated with steel strands (15% versus 29% at 75 years). The GFRP prototype tested proved to be a viable and competitive alternative to traditional (HSCS) and innovative (HSSS, CFRP) PC technologies.

New Solutions

STIC 2018 Proposal *(pending)* – *Basalt-FRP Rebar Standardization*

"Develop standard (guide) design specification, and standard material and construction specifications for basalt fiber-reinforced polymer (BFRP) bars for the internal reinforcement of structural concrete"



FDOT Project Examples

1. **Cedar Key SR24 Bulkhead Rehab.**
   - Construction completed June 2016
   - Construction Project Overview
2. **Halls River Bridge Replacement Project**
   - Letting 6/15/2016
   - FDOT 2015 Design Expo Presentation
   - FDOT 2016 Design Expo Presentation
3. **Bakers Haulover Cut Bridge Bulkhead Rehab.**
   - Letting 6/15/2016
4. **Skyway South Rest Area Seawall Rehab.**
   - Design-Build – 100% Plans
   - Advertisement 04/11/2016

Project Example #1

Cedar Key SR24 Bulkhead Rehab.

Designer: **Kisinger Campo & Associates Corp. (Tampa)**

Structures EOR: **Patrick Mulhearn**

- Replacement of bulkhead cap with GFRP reinforced concrete;
- Addition of Test Blocks on underside of cap with three types of GFRP rebar surface treatments;
- FDOT State Materials Office to perform periodic sampling and monitoring.

KCA KISINGER CAMPO & ASSOCIATES

Design

FDOT

Owner & Maintaining Agency (Bi-Annual Inspection)

U.S. Department of Transportation Federal Highway Administration

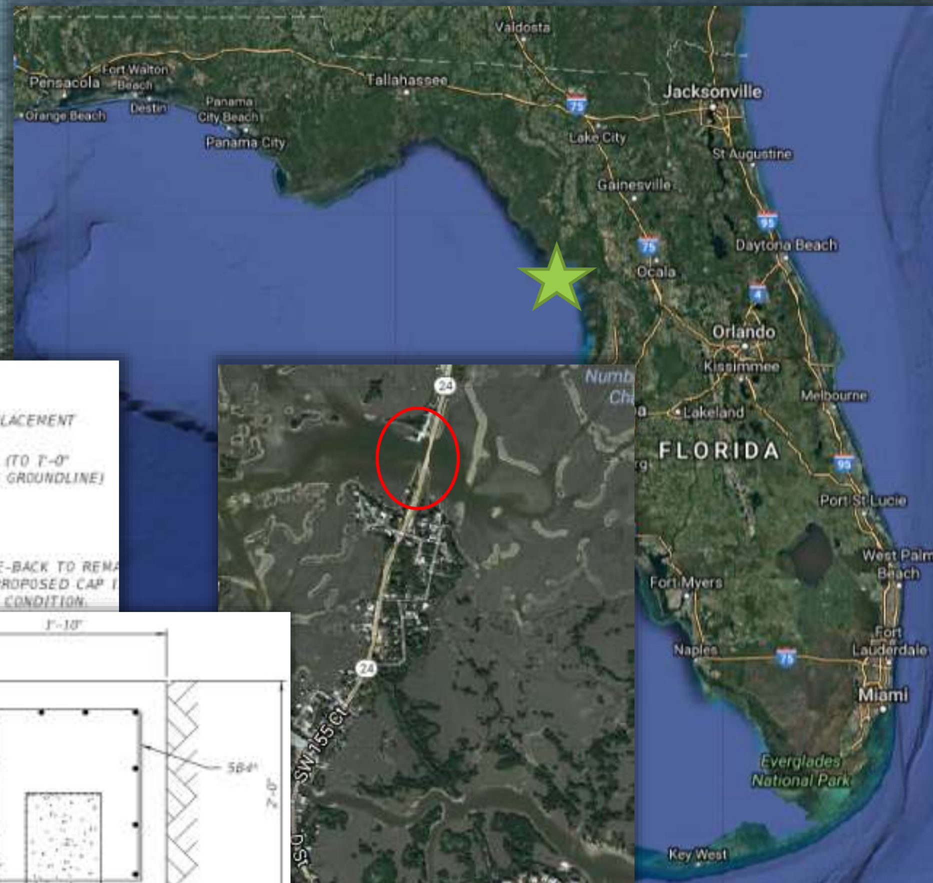
Funding

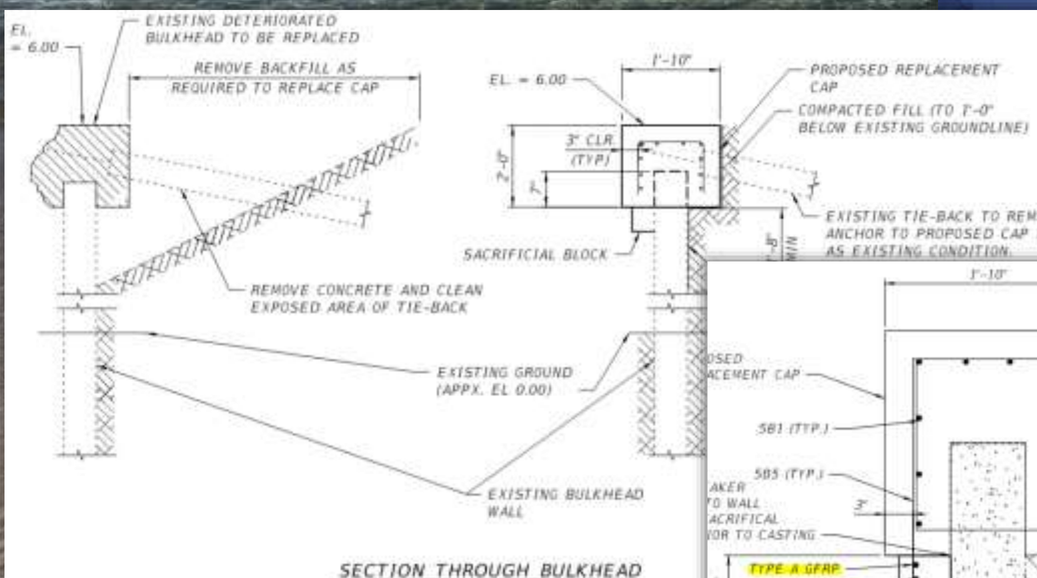
Project Example #1

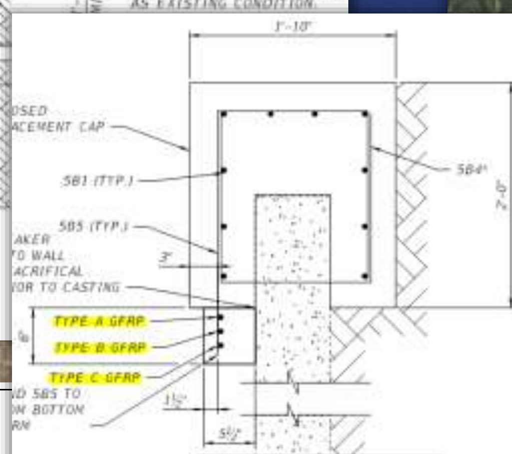
Cedar Key SR24 Bulkhead Rehab.

GFRP BAR TYPE CHART	
TYPE A	SIZE 5 GFRP BAR COATED WITH GRANULAR MATERIAL AND WITH SURFACE INDENTATIONS/DEFORMATIONS CREATED BY HELICAL WRAPPING.
TYPE B	SIZE 5 GFRP BAR COATED WITH GRANULAR MATERIAL AND WITHOUT SURFACE INDENTATIONS/DEFORMATIONS.
TYPE C	SIZE 5 GFRP BAR COATED WITH POLYMER (NO GRANULAR COATING) AND WITH SURFACE INDENTATIONS/DEFORMATIONS CREATED BY SHAPING THE POLYMER COATING (NO EXTERNAL WRAPPING).

TABLE APPLIES ONLY TO REBAR IN SACRIFICIAL BLOCK







914-415-105	GLASS FIBER REINFORCING POLYMER BAR		
...ADS		LF	5088
...ED		LF	131

Project Example #1

Cedar Key SR24 Bulkhead Rehab.


Installing 2-piece stirrup bars in bulkhead cap

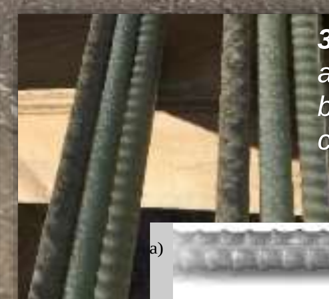

Installing 2-piece stirrup bars in bulkhead cap


Plastic zip-ties for securing GFRP rebar


Curing concrete bulkhead cap prior to form removal


Temporary UV protection for bulkhead cap reinforcing


Forming bulkhead cap



***3 bar-surface types**:*
a) Ribbed
b) Sand-coated
c) Helically wrapped and sand-coated

Project Example #2

Halls River Bridge Replacement

Designer: FDOT District 7 Structures Design Office

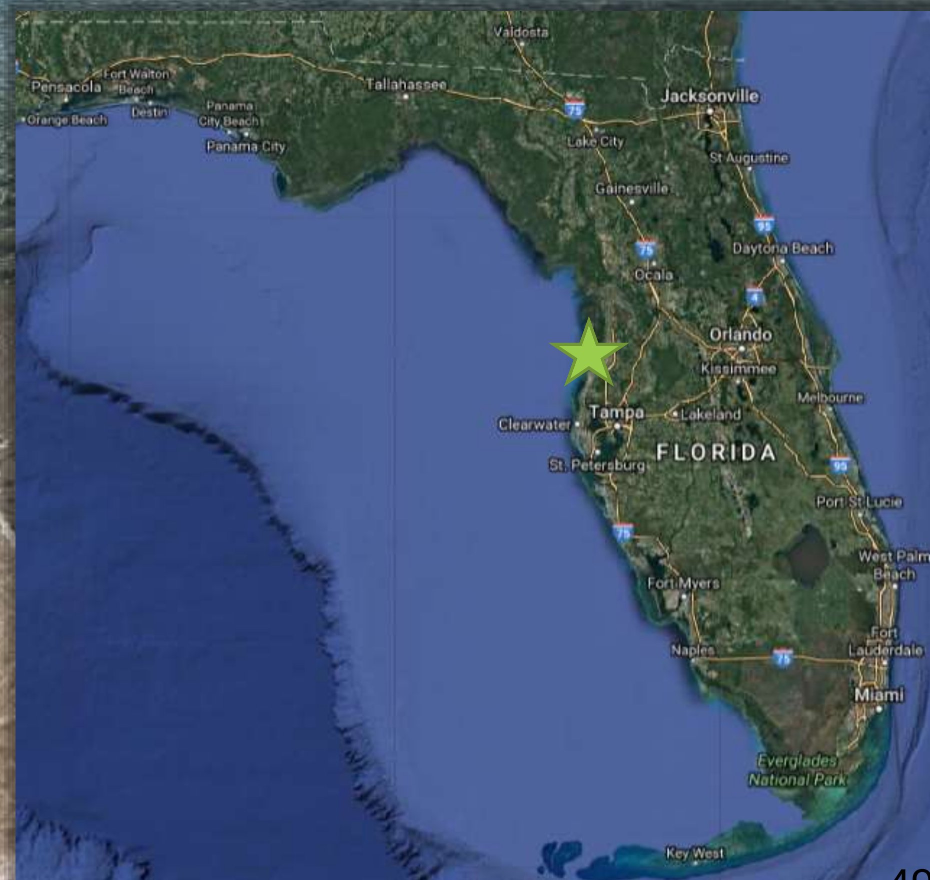
Structures EOR: Mamunur Siddiqui, P.E.

Owner & Maintaining Agency

Design & Bi-Annual Inspection

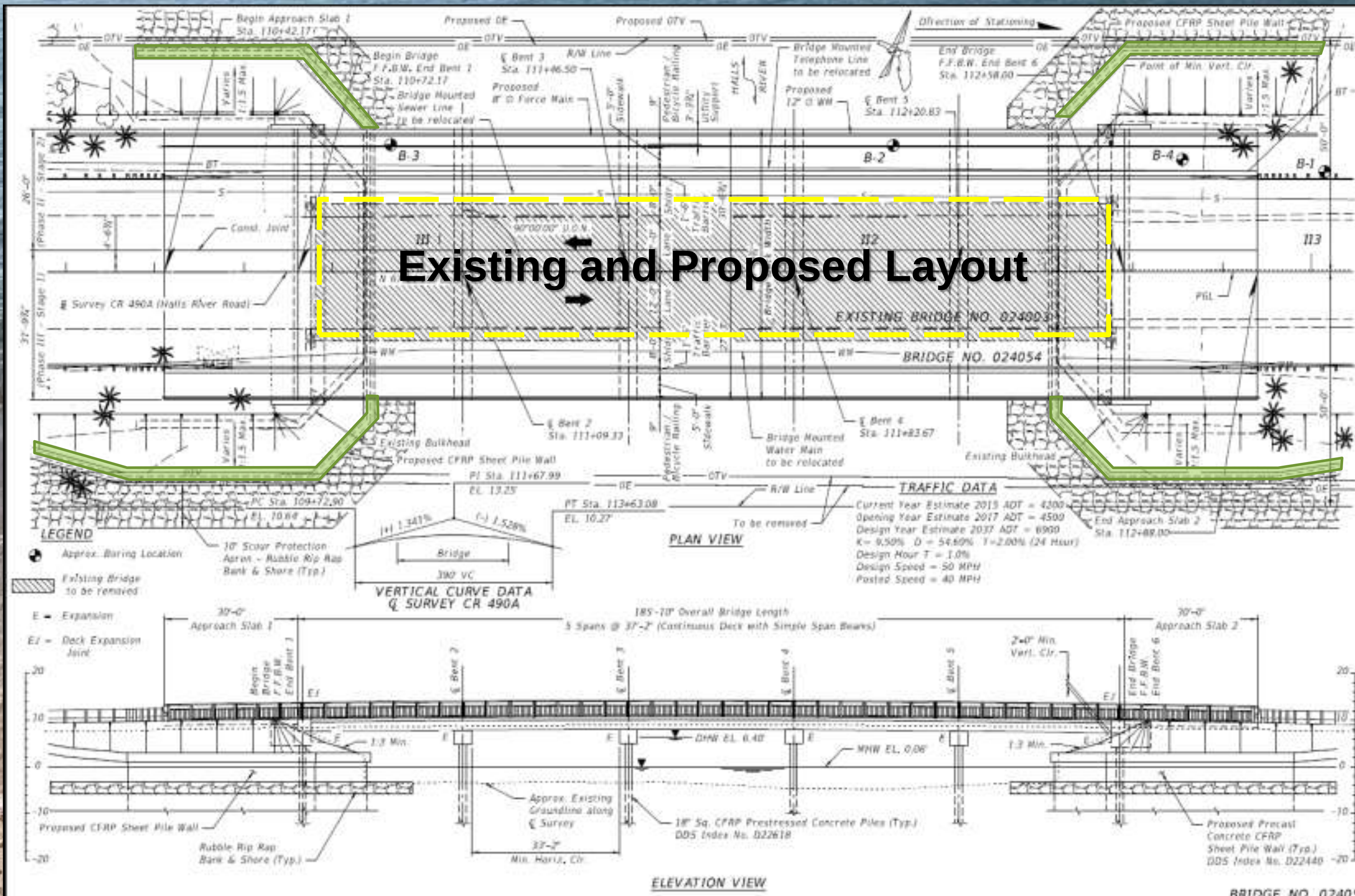
Collaboration Research

Funding & Monitoring



Project Examples #2

Halls River Bridge Replacement



Project Examples #2

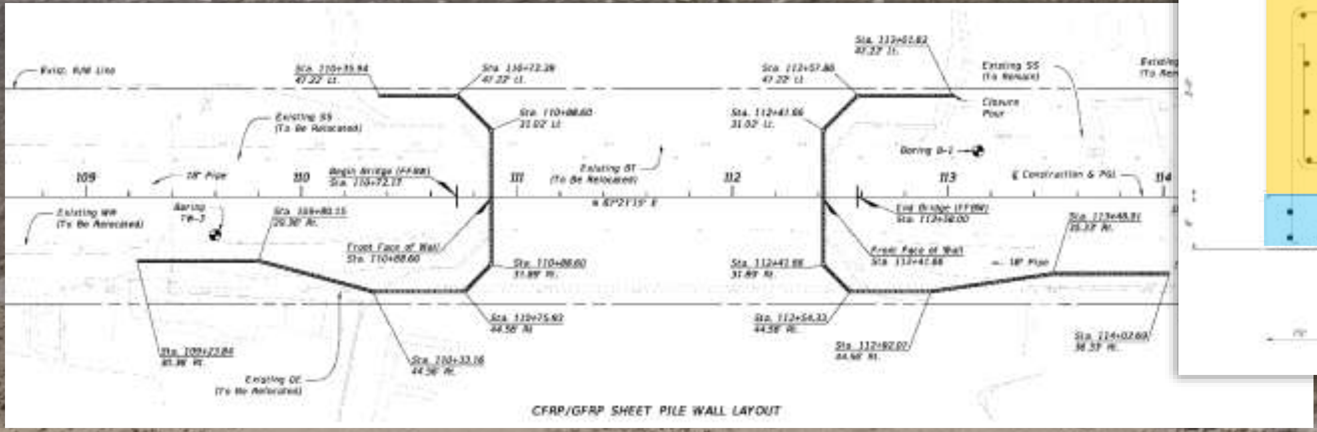
Halls River Bridge Replacement

Precast Prestressed Sheet Piles:

- 12”x30” Steel Reinforced : **$ 120 / ft.**
- 12”x30” CFCC Reinforced : **$ 144 / ft.**

(bid cost was $265)

CFRP/GFRP Sheet Pile Walls



GFRP

CFCC

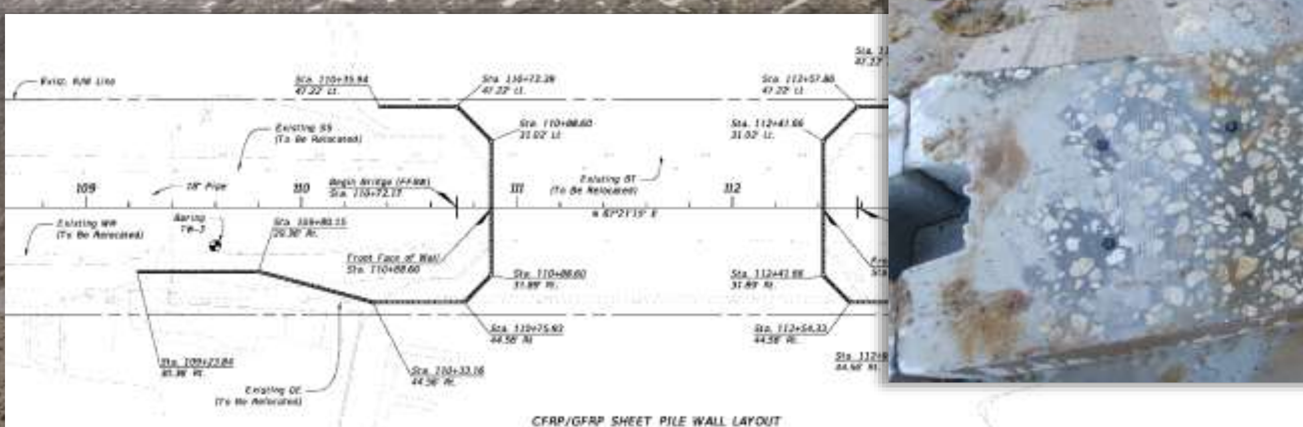
Test Blocks

Monitoring Test Blocks
(To be periodically removed from under Bulkhead Cap)

Project Examples #2

Halls River Bridge Replacement

CFRP/GFRP Sheet Pile Walls



Project Example #3

Bakers Haulover Cut Bridge Bulkhead Replacement

Designer: **Bolton Perez & Associates (Miami)**

Structures EOR: **Joaquin Perez**

Bolton Perez & Associates
Consulting Engineers

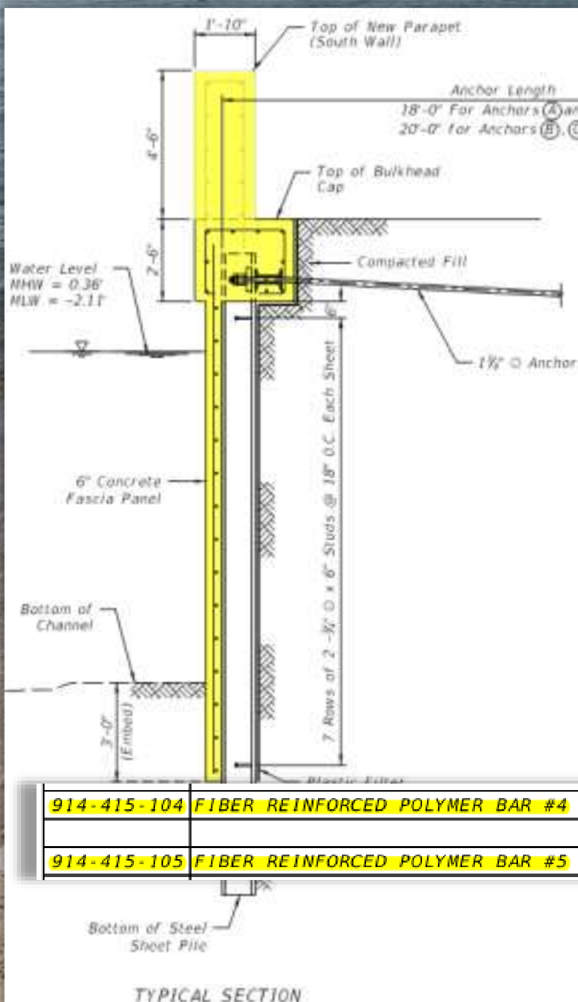
- GFRP Reinforced concrete facing, cap and parapet on a steel sheet pile wall;
- No test blocks.

Owner & Maintaining Agency (Bi-Annual Inspection)

Funding

Project Example #3

Bakers Haulover Cut Bridge Bulkhead Replacement



914-415-104	FIBER REINFORCED POLYMER BAR #4	BULKHEAD CAP, PARAPET & FASCIA PANEL	LF	12,199.32
914-415-105	FIBER REINFORCED POLYMER BAR #5	BULKHEAD CAP, PARAPET & FASCIA PANEL	LF	7,071.14

Project Example #4

Skyway South Rest Area Seawall Rehabilitation

Designer: ***RS&H***

Structures EOR: ***Justin P. Wellborn***

RS&H

(Design-Build-Designer)

FDOT

Owner & Maintaining Agency (Bi-Annual Inspection)

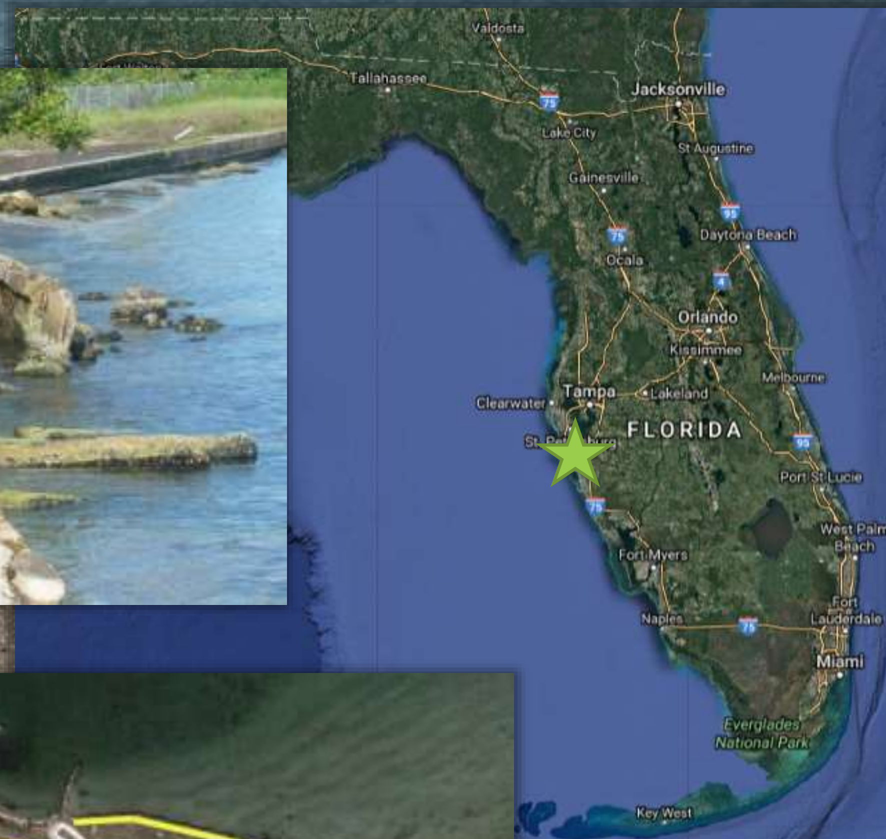
U.S. Department of Transportation Federal Highway Administration

Funding

- Replacement of bulkhead cap with GFRP reinforced concrete;
- CFRP/GFRP Prestressed Concrete Sheet Piles (extension);
- GFRP-RC Traffic Railing;
- No test blocks.

Project Example #4

Cracking of existing seawall bulkhead cap



Limits of seawall bulkhead cap replacement

Limits of seawall bulkhead cap replacement near Rest Area

QUESTIONS ??

Contact Information:

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FDOT's Fiber-Reinforced Polymer Deployment Train

??? → GFRP Prestressing → GFRP Reinforcing Bars → CFRP Prestressing → Navigation Fender Systems → External FRP Laminate Repairs