

Design of Marine Dock Using Concrete Mixed with Seawater and FRP Bars

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Abstract: This paper discusses the design of a marine dock in Miami, Florida reconstructed after sustaining damage from Hurricane Irma in 2017. This pioneering structure implements accelerated bridge construction methods using prefabricated bridge elements and systems. The structure is made of precast concrete members with basalt fiber-reinforced polymer and glass fiber-reinforced polymer bars, and concrete mixed with seawater. The structure is designed to withstand hurricane loading and the choice of materials is intended to leverage sustainability and resilience. DOI: [10.1061/\(ASCE\)CC.1943-5614.0001100](https://doi.org/10.1061/(ASCE)CC.1943-5614.0001100). © 2020 American Society of Civil Engineers.

Introduction

Structures in coastal environments are generally made of steel, wood, or reinforced concrete (RC). RC and prestressed concrete (PC) provide higher strength and durability than wood and are more cost-effective than steel (Saba 2013). However, conventional RC/PC structures are susceptible to corrosion in coastal areas (Dauji 2018). The chloride ions present in seawater can diffuse through the concrete cover and, when they reach the reinforcement, can disrupt the passivation layer and initiate steel corrosion. Corrosion is a limiting factor on the service life of coastal structures (Dauji 2018) and a variety of measures can be implemented to mitigate its effects. One approach to minimize corrosion is to increase concrete cover and use concrete mixes with a lower water–cement ratio and admixtures (FEMA 2013). However, these methods can only delay the process of corrosion, they do not eliminate the problem.

In coastal structures, corrosion can start early in their service life. Costa and Appleton (1999) observed that the deterioration rate of a coastal structure is dependent on its exposure condition, and recorded corrosion rates as high as 500 $\mu\text{m}/\text{year}$ in the tidal zone. One of the main concerns with corrosion is that it causes concrete to spall following the formation of oxides and expansion of the corroding steel. Therefore, more reinforcement is exposed

and the process is accelerated. As corrosion proceeds, the effective area of the reinforcement reduces, negatively affecting the structural capacity of the member (Bilcik and Holly 2013). As a result, coastal structures have relatively short service lives of approximately 30 years (Li 2017) and, when corroded, can be vulnerable to wind–wave loading, particularly during hurricane events.

A perfect example of these phenomena is a dock located in Miami, Florida, which was damaged by Hurricane Irma in 2017 and resulted in the *iDock* reconstruction project—the subject of this paper. The *iDock* is a full replacement of a damaged dock with the implementation of innovative materials and construction methods. This structure pioneers the deployment of accelerated bridge construction (ABC) methods using prefabricated bridge elements and systems (PBES) in coastal structures. The *iDock* is made of precast concrete members partially mixed with seawater and reinforced with basalt fiber-reinforced polymer (BFRP) bars and glass fiber-reinforced polymer (GFRP) bars.

The original dock was built in 1984 with an RC substructure composed of piles and pile caps and a wood deck. During Hurricane Irma, the wood deck was destroyed and only the deteriorated concrete substructure was left in place, as shown in Fig. 1. To facilitate permitting through government agencies and to limit environmental impact, the configuration of the *iDock* mimics the layout and keeps the same footprint as the previously existing dock. However, to provide a more durable structure, the wooden deck was replaced with RC precast slabs carrying a GFRP grating. This solution is in compliance with the Miami-Dade County Department of Environmental Resources Management (DERM) Class I Permit (Miami-Dade 1997) and provides adequate sunlight to seagrass, as shown in Fig. 2.

To eliminate reinforcement corrosion and improve its durability, the *iDock* is entirely reinforced with fiber-reinforced polymer (FRP) bars. FRP is proven to be a durable and noncorrosive reinforcement solution for RC structures (Benzecry et al. 2019) and has been successfully deployed in precast elements (Spadea et al. 2018). Although the cost of FRP reinforcement is higher than the cost of steel reinforcement, the use of FRP in RC/PC structures requires less maintenance and fewer repairs and is projected to provide longer service life than traditional steel reinforcement. The long-term economic benefits of using FRP was found to have a life cycle cost 25% lower than the life cycle cost of using steel for the same RC/PC structure (Cadenazzi et al. 2019).

To demonstrate the use of different reinforcement solutions, BFRP and GFRP bars from three different manufacturers were used in the *iDock*. The RC non-prestressed piles used four different

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Note. This manuscript was submitted on December 3, 2019; approved on September 17, 2020; published online on November 20, 2020. Discussion period open until April 20, 2021; separate discussions must be submitted for individual papers. This paper is part of the *Journal of Composites for Construction*, © ASCE, ISSN 1090-0268.



Fig. 1. Dock after Hurricane Irma. (Image by Vanessa Benzecry.)



Fig. 2. *iDock*: RC structure with GFRP grating. (Image by Vanessa Benzecry.)

reinforcement configurations to demonstrate different constructability approaches and to investigate their ability to provide the required structural capacity. The use of noncorrosive FRP reinforcement eases some of the restrictions to the concrete mix allowing the use of seawater in the mix (Khatibmasjedi et al. 2016). The use of seawater had negligible influence on the material cost but was used as a sustainable alternative to the use of freshwater. Freshwater is a scarce resource in some areas of the world and concrete is estimated to use 1 BCM of water per year (Meyer 2004).

The use of seawater in the concrete mix has shown comparable performance in compressive strength compared with conventional concrete (Khatibmasjedi et al. 2017). Moreover, the use of concrete mixed with seawater and FRP reinforcement may provide environmental benefits. A life cycle assessment comparing identical structures made of steel-RC/PC and FRP-RC/PC with partial use of

seawater in concrete, showed that over a period of 100 years, the FRP-RC/PC alternative produces fewer environmental impacts in four out of five categories. The four categories in which FRP-RC/PC was superior than steel-RC/PC were global warming, eutrophication, acidification, and photochemical oxidant creation. Although ozone depletion was the category that had a higher impact using the FRP-RC/PC alternative, this resulted from the manufacturing process of the material and may not be representative of the current state-of-the-practice (Cadenazzi et al. 2019, 2020). To allow for future durability studies and comparison with conventional concrete, concrete mixed with seawater was used in half of the PBES, while the remaining half of the structure used conventional concrete. The entire structure was specifically designed to implement ABC techniques and increase the capacity of the dock to sustain the impact of major hurricanes.

Structural Configuration for Accelerated Bridge Construction

The *iDock* implemented ABC, which uses innovative planning, design, materials, and construction methods in a safe and cost-effective manner to reduce the onsite construction time for building new structures or replacing and rehabilitating existing ones (Culmo et al. 2011). This technique has seen widespread deployment in bridge construction but is not common in coastal structures. ABC positively affects a variety of factors, such as project delivery time, material quality, and structural durability. It reduces onsite construction time, weather-related time delays and minimizes environmental impacts (Culmo et al. 2011).

To achieve ABC and minimize variations in the layout of the dock due to environmental regulations, the *iDock* was constructed using PBES comprising 8 piles, 4 pile caps, 10 slabs, and a GFRP grating. The use of PBES allows fabrication work to be done ahead of time, which reduces onsite construction time on activities such as formwork preparation, concrete pouring, planning, and the coordination and scheduling of materials and labor. The use of PBES also provides improvements in the safety and quality control of the product, as it is built in a controlled environment (Culmo et al. 2011).

To avoid variations to the footprint of the existing dock, the PBES attempted to be as similar to the existing members as possible. The precast piles are of the same size as the existing dock: 300- × 300-mm square piles for a production length of 7.3 m. The precast pile caps also have the same length and depth, but the width had to be increased to fit the precast piles, resulting in a size of 2,440 × 760 × 300 mm. Furthermore, the precast slabs depth is the same as the existing wood decking (200 mm) and the slabs span between bent caps, which are approximately 3 m apart. The width of the precast slab is 1.2 m as the slabs run on both edges of the dock with the grating in the middle. Fig. 3 shows a cross section through the *iDock*.

The PBES allow rapid construction as the pieces can be simply assembled on site. The construction sequence is shown in Fig. 4. Onsite operations started by driving the precast piles into the pre-existing holes formed by the extracted existing piles. The piles had to have a slight batter (1:20) to fit into the pile caps. After the piles were installed, each cap was lifted by the crane and placed on top of two piles. The slabs were then lifted and placed spanning between caps. For the pile shown in Fig. 5, a post-tensioning (PT) duct of approximately 50 × 20 mm was placed in the center of the pile for a length of 3 m from the top of the pile. The PT duct was used to provide an opening in which to insert dowels and epoxy grout on site. The dowels are approximately 960 mm in

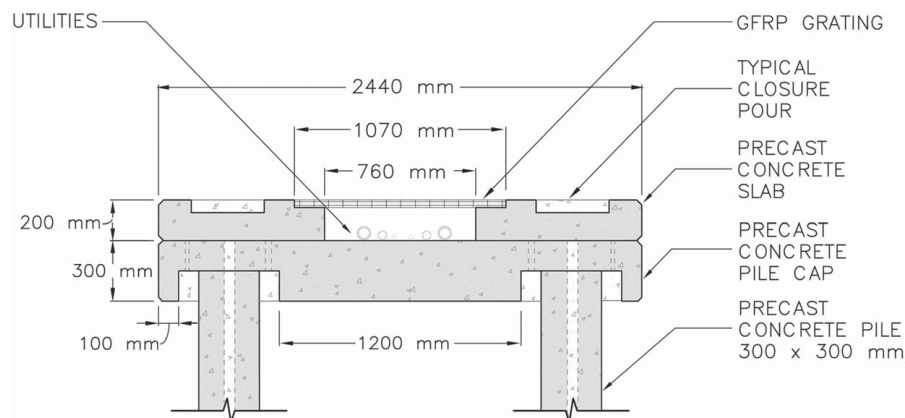


Fig. 3. *iDock* cross section.

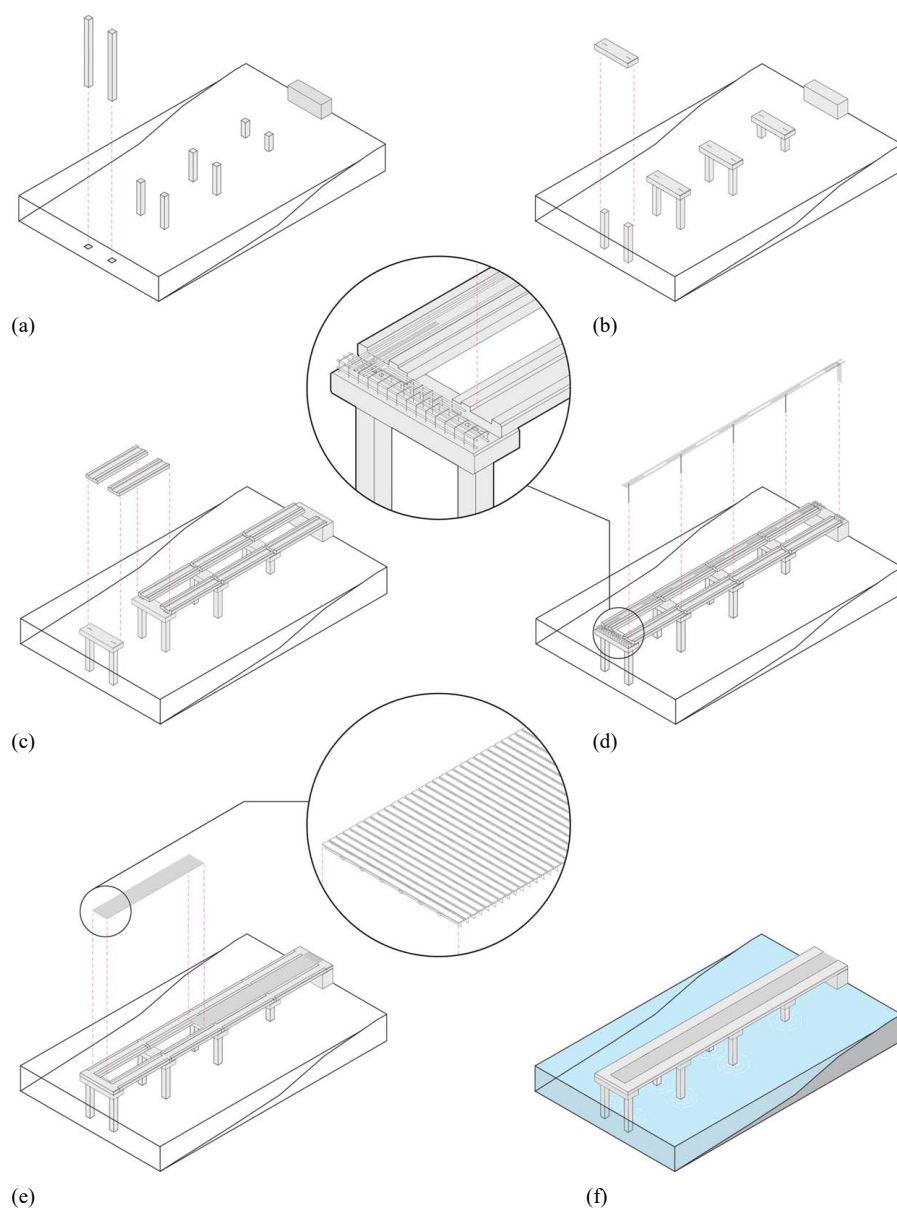


Fig. 4. Accelerated construction sequence: (a) precast pile installation; (b) precast pile cap installation; (c) precast slab installation; (d) placement of bars prior to concrete pour; (e) placement of GFRP grating; and (f) finalized dock.

length, but the PT duct was oversized in case the pile needed to be cut after driving.

Once the pieces are assembled, suitable structural connections between elements are required. GFRP dowels were inserted inside the PT ducts and epoxy grouted. The dowels pass

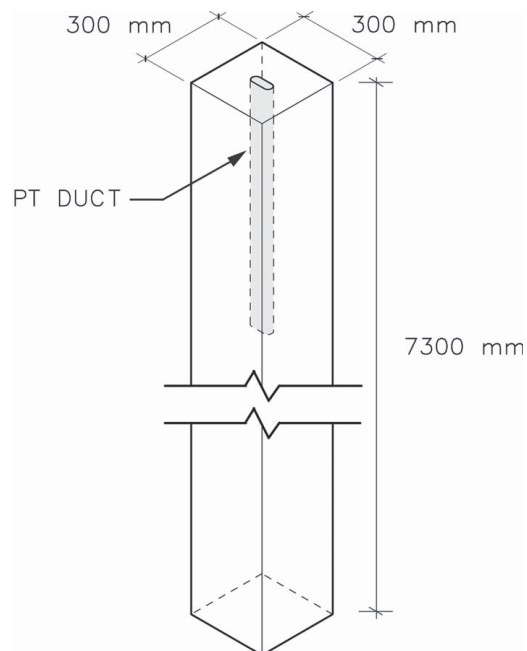


Fig. 5. Pile geometry.

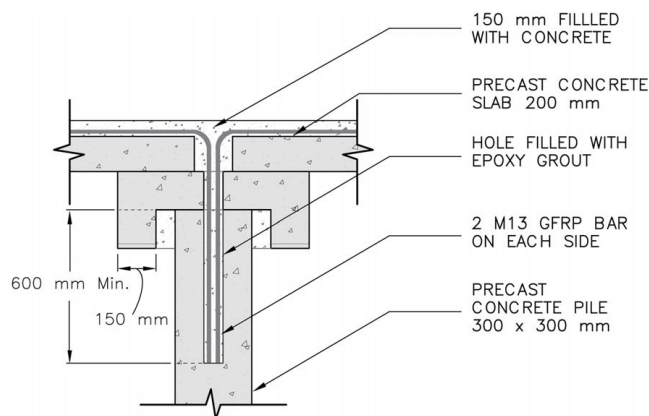


Fig. 6. Pile-pile cap-slab connection detail.

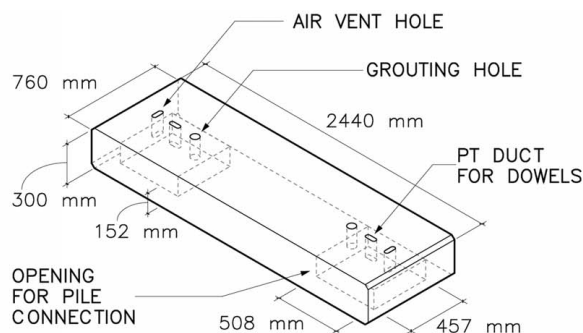
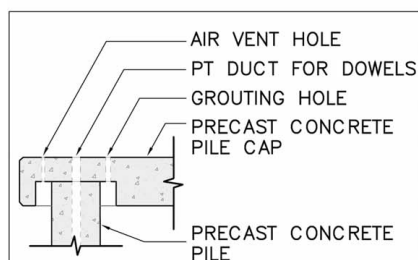


Fig. 7. Pile cap geometry.

through the pile and pile cap and extend in two directions into the recesses in the slabs, as shown in Fig. 6. Spliced BFRP bars and a BFRP mesh were placed in the slab recesses along the entire length of the dock in preparation for the second pour. Dowels, positioned longitudinally, and transverse bars were placed at the end of the dock to assure proper connection between precast and cast-in-place concrete, as shown in the detail (d) of Fig. 4. Finally, concrete was poured into the recesses and openings between the elements to provide proper connection between all the elements.

For the pile cap shown in Fig. 7, openings were needed to assemble the pile cap on top of the piles, and to connect the pile cap to the piles and slabs. Each pile cap has two sockets of $508 \times 457 \times 152$ mm to fit the top of the piles, located at approximately 100 mm from its edge. Three PT ducts were also placed directly above the pile opening. One of the PT ducts was the same as used in the pile, and was placed directly above the pile PT duct to provide a continuous opening for the dowels to go from the pile through the pile cap. A round PT duct with a diameter of 57 mm was used as a passage through which to pour epoxy grout around the pile and close the opening. The use of formwork around the pile was necessary to contain the grout, and the third PT duct, of an oval shape of approximately 50×3 mm, was used to allow air to escape during the pouring of the grout.

The slabs measure 840×200 mm, as shown in Fig. 8, and span from pile cap to pile cap. Due to the dock being slightly angled, and the distance between pile caps varying, the slabs have lengths of 3.65, 3.35, and 3.00 m. To connect the slabs to the piles and pile caps and to assure continuity, an opening of approximately 150 mm was left between the slabs, and a recess of 63×370 mm with a roughened concrete finish was formed in the slab for placement of bars and concrete. The recess ran through the entire length of the slab and therefore allowed the placement of bars along the entire length of the dock for continuity at the joints and to provide capacity against uplift.

Materials and Reinforcement Layout

The *iDock* was designed and constructed using state-of-the-practice noncorrosive FRP bars and concrete mixed with seawater. The piles are non-prestressed and have four different reinforcement configurations to investigate provision of the required structural capacity. In addition, to leverage sustainability and for future durability investigation, half of the PBES of the *iDock* were made with seawater concrete, while the remainder were made with conventional concrete. The connections between the elements either used conventional concrete or epoxy grout.

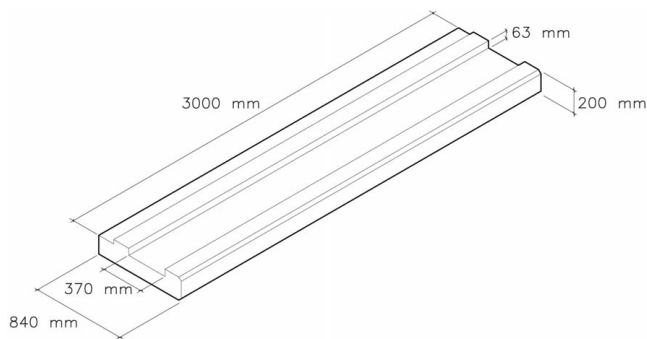


Fig. 8. Slab geometry.

Table 1. FRP bars properties

FRP type	Nominal diameter (mm)	Manufacturer	Tensile strength (MPa)	Modulus of elasticity (GPa)
BFRP Mesh	2.2	C	1,200	59
GFRP	10	A	1,100	60
GFRP	10	B	827	46
GFRP	13	A	1,100	60
GFRP	13	B	758	46
GFRP	19	A	1,100	60
GFRP	19	B	690	46
BFRP	19	C	1,303	59
GFRP	25	A	1,100	60

Table 2. Conventional precast mix design

Material	Quantity	Unit
Cement Type IL	323	kg/m ³
Fly ash	81	kg/m ³
No.57 stone	979	kg/m ³
Silica sand (concrete)	728	kg/m ³
Water	152	kg/m ³
Air entraining (AE 90)	74	g/m ³
Retarder (961 R)	1,261	g/m ³
Water reducer (Glenium 7920)	630	g/m ³

Reinforcement

The piles and pile caps are reinforced with GFRP bars from two different manufacturers (designated A and B in this study), whereas the slabs were reinforced with BFRP bars from a third manufacturer (designated C). The FRP bars used in the *iDock* had sand-coated surface treatment and their properties are presented in Table 1 as reported by each manufacturer. The properties used for design were as specified by ASTM D7957 (ASTM 2017) irrespectively of the manufacturer.

The GFRP bars used for the piles were manufactured by A. The reinforcement consisted of M19 and M25 straight bars and M10 stirrups and spirals. The stirrups were made of two (2) C-shaped bars zip-tied together. For the pile caps, the bars used were manufactured by B, and consisted of M19 straight bars, M13 bent bars (C-shaped and U-shaped) and M10 stirrups. To assure a proper connection between pile, pile cap, and slabs, M13 L-shaped bars were also used. The BFRP bars used for the slabs were manufactured by C. The bars consisted of M19 straight bars and a M6 square mesh with openings of 50 × 50 mm.

Concrete

The three concrete mixes chosen for the *iDock* were designed for marine applications. The mix designs are in accordance with Class IV requirements as defined by Florida Department of Transportation (FDOT) specifications for road and bridge construction (FDOT 2017) that require a compressive strength of at least 38 MPa at 28 days.

Conventional Concrete – Precast Elements

The traditional concrete mix for the precast elements used Type II Portland limestone cement and fly ash. The quantity of Portland cement to be replaced with fly ash was 20% of the total weight of cementitious materials. The concrete mix included No. 57 stone aggregates and required a maximum water to cement ratio of 0.38. The measured concrete slump was 220 mm. The concrete mix is given in Table 2 and compressive strength test results at different times are shown in Fig. 9. The compressive strength test was performed in accordance with ASTM C39/C39M (ASTM 2018).

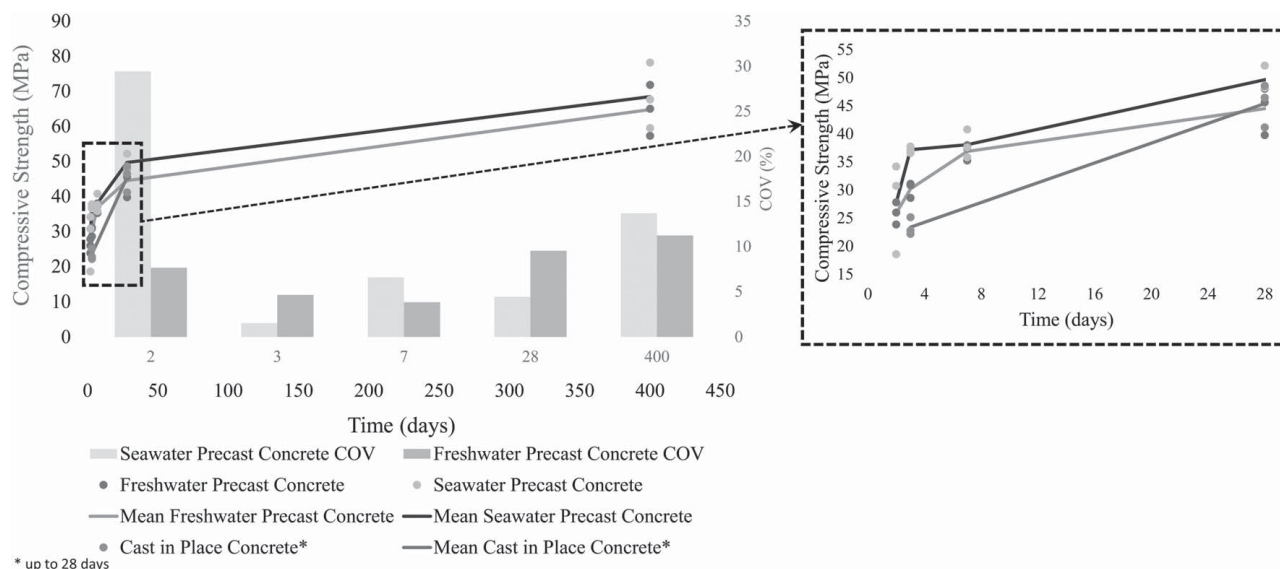


Fig. 9. Concrete compressive strength results.

Three specimens, 100 mm in diameter $\times$ 200 mm in length, were tested at each time period.

Seawater Concrete – Precast Elements

To allow for a direct comparison, the seawater mix used for the precast elements was designed as in the conventional mix, except potable water was substituted with seawater and the quantity of retarder was doubled. The measured concrete slump was 240 mm. The concrete mix is presented in Table 3 and the compressive strength test results at different times are shown in Fig. 9.

Conventional Concrete – Cast-in-Place

Due to the relatively shallow depth (63 mm) of the slab recesses used to develop the connection between the elements of the *iDock*, the design mix for the cast-in-place concrete required smaller aggregates. Instead of using No. 57 stones as for the precast mixes, No. 89 stones were used. The cast-in-place mix design used Type II (MH) cement and slag as approximately 52% of the cementitious material. The water to cement ratio was 0.40. The measured concrete slump was 248 mm. The concrete mix is given in Table 4 and the compressive strength test results up to 28 days are shown in Fig. 9. The average 28-day compressive strength was 45 MPa with a coefficient of variation of 8.4%.

Loading Conditions

As the existing dock was damaged by Hurricane Irma when it landed as a Category 2 hurricane in Miami-Dade County, it was decided to consider loads associated with hurricane conditions in the new design. As wave loads are the main cause of damage in coastal structures during storms (FEMA 2013), to build a resilient dock, not only did the materials used have to be optimized but the load demands also had to be calibrated accordingly.

Wind and wave loads associated with a 100-year storm were considered in the design of the *iDock*. The wind loads were obtained from ASCE 7-10 (ASCE 2010), and the wave loads were analyzed considering three different contributions: uplift, lateral wave

load, and lateral wave slam. The lateral wave load and wave slam were calculated using the *Coastal Construction Manual* (CCM) (FEMA 2011) and the uplift wave load was calculated from the coastal engineering manual (US Army Corps of Engineer 2011).

In addition to wind and wave loading, and considering the potential upscaling of the technology, the dock was designed for vertical loading compatible with the design of a pedestrian bridge according to AASHTO *LRFD Guide Specifications for the Design of Pedestrian Bridges* (AASHTO 2009).

Wind Load

The wind loads for a Category 5 hurricane in South Florida were obtained from ASCE 7-10 (ASCE 2010). The wind speed was obtained from the wind speed figure for a Risk Category II structure and indicates a structure that may present some risk to human life in case of failure. The wind speed obtained was $V_u = 282$ km/h.

The wind pressures were calculated using the envelope procedure main wind force resisting system, used for low-rise buildings, which considers wind loads in different directions: windward, leeward, and side walls. The calculations considered a partially enclosed building with an Exposure D category, which indicates structures that are located in unobstructed areas or on water surfaces. The highest wind pressures obtained were a suction pressure of $p = 5.36$ kN/m² on horizontal components and a pushing toward pressure of $p = 3.13$ kN/m² on vertical components.

Wave Load

The first step to calculating wave loads was to obtain the design stillwater flood elevation (E_{sw}), the bay bottom depth to calculate the design stillwater flood depth (d_s), and the breaking wave height (H_b). E_{sw} was obtained from the FEMA Flood Insurance Study (FEMA 2009), Transect 22, based on a 100-year storm event, and the bay bottom elevation was obtained through a field inspection. The elevations are based on the National Geodetic Vertical Datum (NGVD 29). The design stillwater flood depth and the breaking wave height, shown in Fig. 10, were calculated using Eqs. (1) and (2) from the CCM (FEMA 2011).

$$d_s = E_{sw} - GS \quad (1)$$

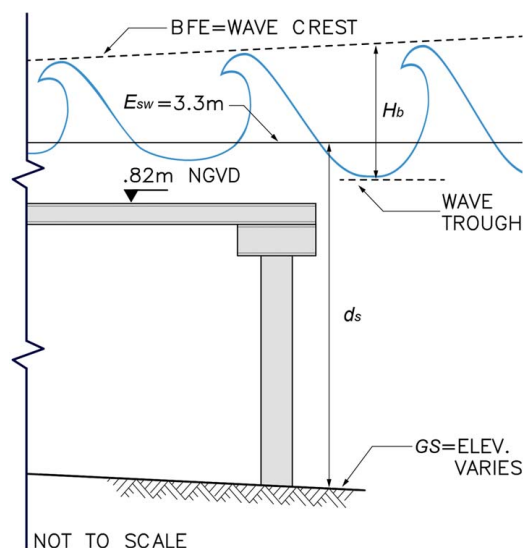


Fig. 10. Design flood conditions.

Table 3. Seawater precast mix design

Material	Quantity	Unit
Cement Type I/II	323	kg/m ³
Fly ash	81	kg/m ³
No. 57 stone	979	kg/m ³
Sand	728	kg/m ³
Seawater	152	kg/m ³
Air entraining (AE 90)	74	g/m ³
Retarder (961 R)	2,522	g/m ³
Water reducer (Glenium 7920)	630	g/m ³

Table 4. Conventional concrete mix design

Material	Quantity	Unit
Cement Type II (MH)	214	kg/m ³
Slag	228	kg/m ³
#89 Stone	838	kg/m ³
Silica sand (concrete)	793	kg/m ³
Water	178	kg/m ³
Air entraining (Isosphere 5004)	37	g/m ³
Retarder (Isoplast 1440)	1,334	g/m ³
Water reducer (Isoflow 8530)	408	g/m ³

$$H_b = 0.78d_s \quad (2)$$

where GS is the lowest eroded ground elevation, in meters above datum, adjacent to a building, excluding the effects of localized scour around the foundation.

The breaking wave height was 2.96 m. With the breaking wave height and the design stillwater flood depth, the breaking wave force could be calculated. Eq. (3) from CCM was used to calculate the breaking wave force on the 300 mm square piles.

$$F_{brkp} = \left(\frac{1}{2}\right) C_{db} \gamma_w D H_b^2 \quad (3)$$

where F_{brkp} = breaking wave force (kN) acting at the stillwater elevation; C_{db} = breaking wave drag coefficient (2.25 for square piles); γ_w = specific weight of water (997 kg/m³ for saltwater); D = 1.4 times the width of the pile or column for a square pile; and H_b = breaking wave height.

The breaking wave force on one pile was found to be 42.42 kN. To calculate the wave slam on the pile cap, the height of the wave crest is needed to obtain h (the distance between the wave crest and the bottom of the floor beam) in Eq. (4). The wave crest elevation is the same as the base flood elevation (BFE). The FEMA flood zone of the project was found in the FEMA flood map (FEMA 2009) to be VE, where flood insurance is mandatory as it indicates an area of high risk of flooding (more than 1% chance per year) and additional hazard associated with storm waves. The corresponding value for the BFE was taken from the FEMA Flood Insurance Study (FEMA 2009) for Transect 22 and flood zone VE and was found to be 5 m NGVD.

In this case, because more than 70% of the breaking wave height is above the top of the dock, and the wave will dissipate beyond the face of the dock, h was taken as the height of the floor beams to calculate the lateral wave slam.

$$F_s = f_s w = (1/2) \gamma_w C_s h w \quad (4)$$

where F_s = lateral wave slam (N); f_s = lateral wave slam (N/m); C_s = slam coefficient for typical residential structure (2.0); h = wave crest extension (m) above the bottom of the floor joist or beam; and w = length (m) of the floor joist or floor beam struck by wave crest.

Using Eq. (4), the wave slam on the pile cap was found to be approximately 71 kN. The uplift force was calculated using the Goda formula for irregular waves (CEM Table VI-5-53) from the coastal engineering manual (US Army Corps of Engineer 2011). The uplift force (F_u) on the pile cap was found to be 12.6 kN/m and the uplift force on the slabs was found to be 13.9 kN/m. Fig. 11 shows the different wave forces acting on the dock structure.

Live Load

The loads for a pedestrian bridge in accordance with the AASHTO LRFD Bridge Design Specifications (AASHTO 2017) includes an

H-5 truck load, which corresponds to 25% of an HL-93 loading and a pedestrian load of 4.3 kN/m².

Dead Load

The only dead load acting on the dock is the self-weight of the elements. The unit weight of concrete was taken as 2,323 kg/m³.

Structural Analysis

A publicly accessible software developed by FDOT was used to analyze the bent cap and piles for lateral stability as a frame. The software is divided into three different parts: (1) load generator, (2) frame analysis and design, and (3) code checks. The load generator calculates the factored loads on the bent cap. The design of a coastal bridge in accordance with AASHTO (2009), includes the following loads: dead load of structural members, dead load of wearing surface and utilities, live load (LL), centrifugal force (CE), braking force, wind load on structure (WS), wind on live loads, water load, and force effect due to uniform temperature (TU). The CE and TU loads were neglected in the design of the *iDock*.

The LLs were generated for each cap using the lever arm rule and tributary area methods. WS included the 3-s gust wind speed with the use of different load combinations: Strength I, Strength III, Strength V, Service I, and Service IV. The LL included the H-5 vehicular load, which is representative of an emergency or maintenance vehicle. The only load case relevant for the *iDock* was the one-lane load plus impact load of one truck (H-5 load). This load was multiplied by the distribution factors and multiple presence load intensity factor (AASHTO 2017). A fatigue LL of one truck was also calculated even though it would not usually be relevant for a pedestrian structure with only the occasional vehicular load.

The values found under the Loading Conditions section were factored using the load factors as per LRFD Table 3.4.1-1 (AASHTO 2017) for strength, service, and extreme event limit states. The load combinations used were: Strength I max vertical, Strength III max vertical, Strength V max vertical, Service I, Service III, Service I DL, Strength I min vertical, Strength III min vertical, Strength V min vertical and Fatigue I. At the end of the first part of the software, the loads for each combination and each direction (x , y and z) were obtained.

In the second part of the software, frame analysis and design, the bent cap was analyzed for two cases: pinned connection and fixed connection of the piles to the bent cap. The stiffness matrix method was used to solve both cases. The forces transferred to the piles and the shear and moment diagrams for the critical sections at the pile cap were computed. The critical design sections for the bent cap were located at the center of the columns for maximum negative bending, at the center of the cap for maximum positive bending moment, and at the face of columns for maximum shear.

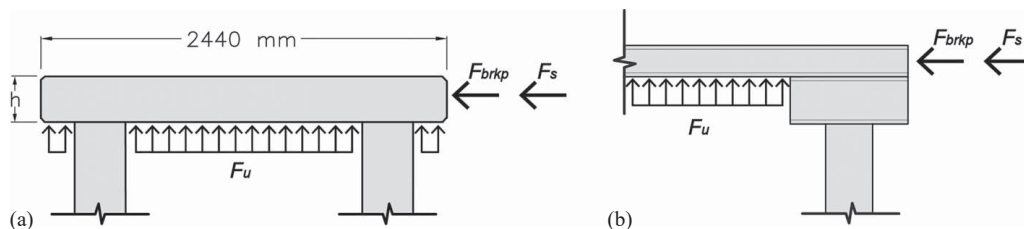


Fig. 11. Wave loads: (a) transverse direction; and (b) longitudinal direction.

The pile top deflections at the ground level were also calculated for the two models (fixed and pinned connection) to determine the minimum pile depth for each model. The results showed that with four non-prestressed GFRP bars connecting the pile to the caps (under both fixed and pinned connection) the minimum pile depth was 3.65 m.

Design Method and Calculations

The design of the *iDock* included the design of the different precast elements individually, the grating, the grating connectors, and the cast-in-place connections between pile and pile cap and slab. The design loads were combined using the strength and service load combinations prescribed by AASHTO (2017). The FRP reinforcement was designed according to the second edition of *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete* (AASHTO 2018) that was under development at the time of design (Rossini et al. 2019b). The methods and results for each component are shown in this section. Further details on the design of FRP-RC elements for bridges, buildings, and coastal structures are discussed by Jawaheri Zadeh and Nanni (2013), Nanni et al. (2014), and Rossini et al. (2018, 2019a, c, d).

BFRP-Reinforced Slabs

To maximize the load demand at each critical section, the slabs were designed to resist positive and negative bending moment at midspan assuming a simply supported configuration. The continuous connections between adjacent spans were also required to resist negative bending moment, computed assuming vertical load acting on both the adjacent spans and fixed connections between adjacent slabs. The load combinations from AASHTO (2017) were adapted to have the pedestrian load decoupled from wave and wind vertical action. The worst-case scenario for gravity and pedestrian loads and uplift loads were used in the design. The concrete clear cover used was 25 mm to the longitudinal reinforcement, in accordance with ACI 440.5-08 (ACI 2008). The results are reported in the following:

- At the bottom layer, 8~M19 BFRP straight bars and M6 BFRP mesh were used to resist positive bending moment from gravity load and pedestrian traffic at midspan. The bars were distributed

over the entire width of the slab, at the bottom. The section was over reinforced (compression-controlled) against positive bending moment. The factored flexural strength was 89.0 kN-m for a flexural demand of 30.0 kN-m and the factored shear strength was 43.6 kN for a shear strength demand of 29.8 kN.

- At the top layer, 8~M19 BFRP straight bars and M6 BFRP mesh were used to resist negative bending moment from uplift loads, at midspan. The bars were distributed over the entire width of the slab, with four of the bars being located in the precast portion, and four being placed in the recess with cast-in-place concrete. The latter extends beyond the length of the single slab for continuity at the joint location, as shown in detail (d) of Fig. 4. The element was designed for flexural failure to occur in the transition zone (between the compression- and tension-controlled strain limits). The factored flexural strength was 96.3 kN-m for a flexural demand of 23.0 kN-m and the factored shear strength was 41.4 kN for a shear strength demand of 23.1 kN.
- At the joint location, 4~M19 BFRP bars were used to resist negative bending moment and assure continuity. This section was under reinforced (tension-controlled). The factored flexural strength was 52.9 kN-m for a flexural demand of 30.0 kN. An isometric view of the slab is shown in Fig. 12.

GFRP-Reinforced Pile Caps

The pile caps were designed to resist positive and negative bending moment at midspan, assuming a simply supported configuration. The load combinations used were service and strength design from AASHTO (2017) and were adapted to have traffic/pedestrian load decoupled from wave vertical action. The worst-case scenario for gravity and pedestrian loads and uplift loads was used in the design. The concrete clear cover used was 38 mm from the transverse reinforcement, in accordance with ACI 440.5-08 (ACI 2008). The results are reported in the following:

- At the bottom layer, 2~M19 straight GFRP bars and 4~M13 U-shaped GFRP bars were used to resist positive bending moment from gravity load and pedestrian traffic, at midspan. The U-shaped bars were distributed over the central portion of the bent cap, at the bottom. This section was under reinforced (tension-controlled) against positive bending moment. The factored flexural strength was 61.0 kN-m for a flexural demand of

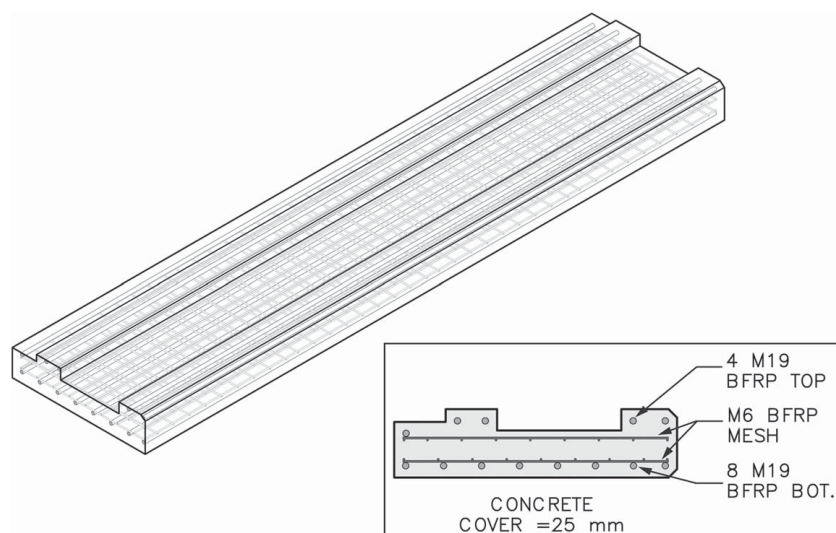


Fig. 12. Slab reinforcement isometric and section view.

30.0 kN-m and the factored shear strength was 71.2 kN for a shear strength demand of 48.5 kN.

- At the top layer, 6~M19 GFRP straight bars were used to resist negative bending moment from uplift loads, at midspan. The bars were distributed over the entire width of the pile cap. The section was under reinforced (tension-controlled) against negative bending moment. The factored flexural strength was 95 kN-m for a flexural demand of 30.0 kN-m and the factored shear strength was 80.0 kN for a shear strength demand of 48.5 kN.
- M10 double C-shaped GFRP stirrups (closed) at 150 mm spacing were used to provide additional shear resistance. At the pile location and around the socket, U-shaped GFRP stirrups (open) were used. In the horizontal plane, at the face of the pile cap, M13 C-shaped GFRP bars were used to provide confinement around the sockets. Shear strengths and demands are detailed in this section. An isometric view of the pile cap is shown in Fig. 13.

GFRP-Reinforced Piles

A total of eight piles with a cross section of 300×300 mm was used in the project. The production length of the pile was 7.3 m but they were cut to length having been driven to refusal on bedrock at approximately 6.1 m, corresponding to the length of the existing piles. The reinforcement had four different configurations to present viable options for the non-prestressed piles:

- 8~M25 longitudinal bars with M10 spirals for two piles,
- 8~M25 longitudinal bars with M10 ties for two piles,
- 8~M19 bars with M10 spirals for two piles, and
- 8~M19 bars with M10 ties for two piles.

The concrete clear cover used was 38 mm from the transverse reinforcement. An isometric (transparent) view of reinforcement options B and C is shown in Fig. 14.

The piles were designed according to AASHTO (2018) as part of an experimental field application to demonstrate the applicability of GFRP longitudinal bars, ties, and spirals as reinforcements for non-prestressed piles. The precast piles located in the front of the dock are the most exposed to wave load and experience a flexural demand equal to 65.1 kN-m. The piles located closer to the coastline experience a reduced impact from waves and wind resulting in a flexural demand estimated at approximately 75% of

the maximum value. The maximum compressive load on each pile is computed as 122.3 kN, whereas the minimum compressive load is equal to 6.9 kN in the situation when uplift almost counterbalances the weight of the pile.

Reinforcement options A and B were used in the front portion of the dock, whereas reinforcement options C and D were used in the portion closer to the coastline. Interaction diagrams and load demands for the four reinforcement configurations are shown in Figs. 15 and 16. The construction of interaction diagrams for GFRP-RC piles followed the same principles discussed by Jawaheri Zadeh and Nanni (2013) and Nanni et al. (2014) for the case of columns. Specifications for the design of GFRP-RC piles were provided by AASHTO (2018).

Pile-Cap-Slab Connection

To assure continuity and provide a connection between the PBES, a PT duct was left open to facilitate the connection between pile, pile cap, and slab. The connecting dowels were epoxy grouted into the PT duct and concrete was cast around them with additional reinforcement for a composite connection. The oval PT duct of approximately 50×20 mm was placed in the center of the precast pile over a depth of 3 m from the top of the pile to provide room for the dowels and epoxy grout. The long length of the PT duct (3 m) was to account for the potential need to cut the top of the pile if driving refusal was encountered prior to the proposed grade. Between the slabs, a gap of approximately 150 mm was left in which to place concrete and extra reinforcement from the piles.

The dowels inserted into the piles were four M13 GFRP bars. To allow the dowels to go through the pile cap and develop a connection with the slabs as shown in Fig. 6, the same PT duct used inside the pile was also placed in the precast pile cap directly above the center of the pile. The dowels were inserted at least 600 mm into the pile PT duct with epoxy grout and had at least 600 mm of development length inside the slab recess. Each dowel was engaged at approximately 30% of its design capacity under maximum uplift. Design capacity was computed at the bent location.

To resist negative bending moment and provide continuity at the location of supports, M19 BFRP bars were placed inside the slab recess. These bars ran over the entire length of the dock and were spliced. A total of 16 bars were used with a minimum splice

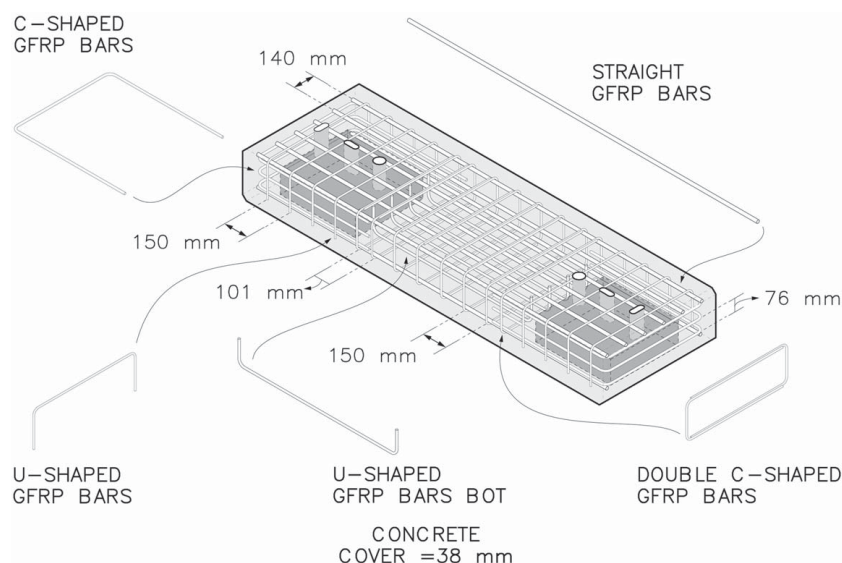


Fig. 13. Pile cap reinforcement isometric view.

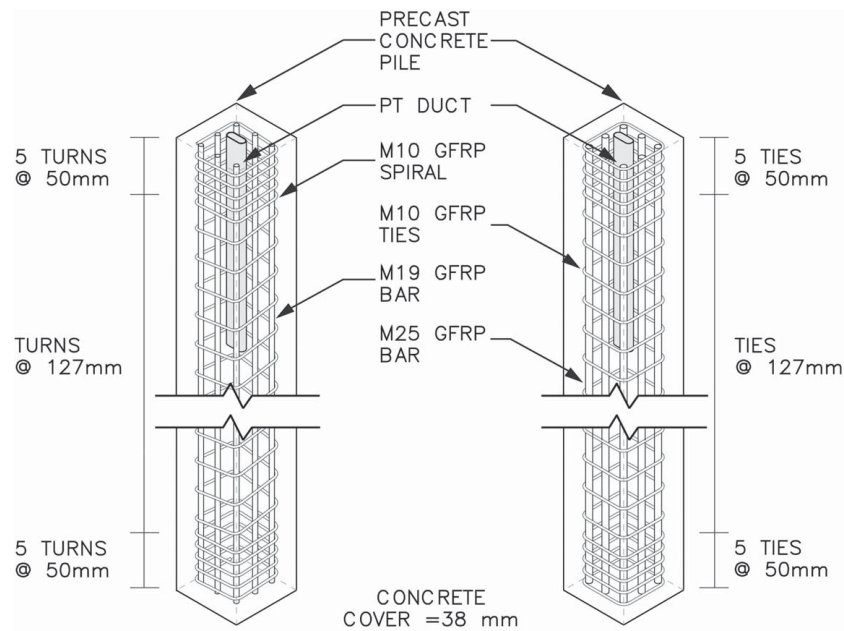


Fig. 14. Pile reinforcement isometric (transparent) view of options B and C.

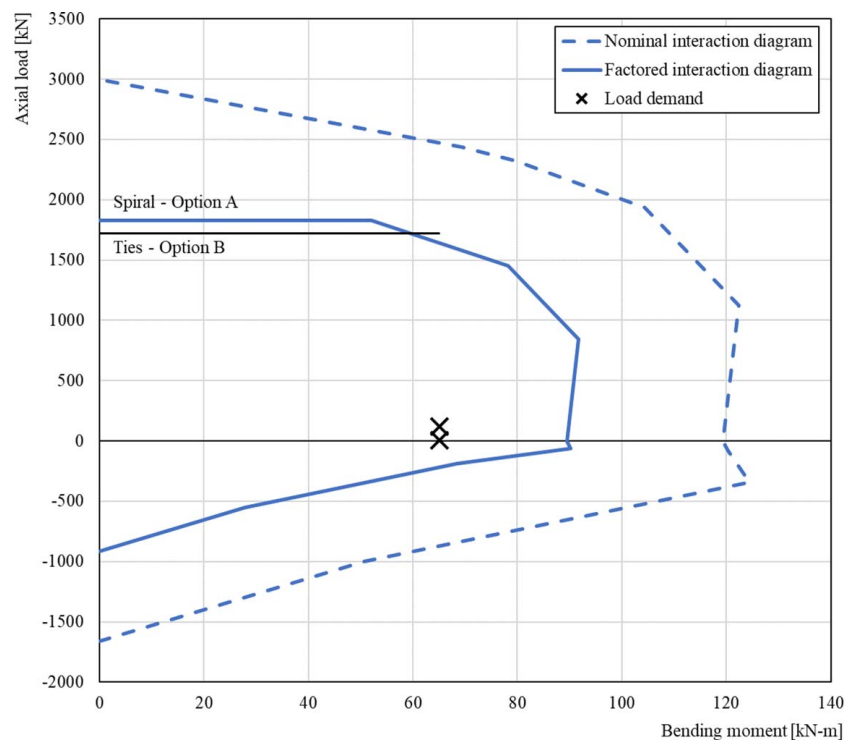


Fig. 15. Pile interaction diagram for options A and B.

of 1.2 m. The recesses and the gap between slabs were then filled with cast-in-place concrete.

Grating

A GFRP grating was chosen for decking purposes. The grating is of a light gray color with a urethane topcoat for extra UV protection, as shown in Fig. 2. The grating size was chosen based on the manufacturer's specification to support pedestrian LL as per AASHTO (2009). The grating selected had 38 mm of bearing

bar thickness and a width (span) of approximately 1,050 mm. This grating can support a uniform load of 4.8 kN/m^2 with a deflection of 2.3 mm.

The grating connectors used were specific for the grating type and size chosen. A hold down insert clip, which consisted of a stainless-steel base plate, was installed with the use of a 6 mm screw anchor into the concrete surface. The quantity of connectors was calculated in accordance with ACI 318-14 Chapter 17 (ACI 2014) with the uplift being the controlling case, which resulted in a minimum of 10 connectors per grating panel (1.5 m long).

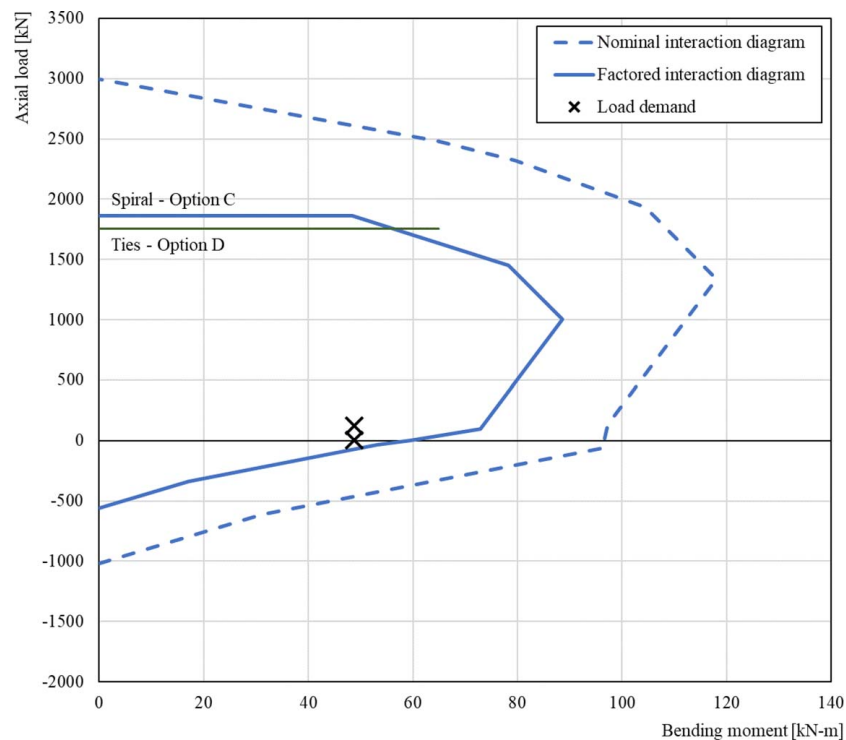


Fig. 16. Pile interaction diagram for options C and D.

Implications for the State-of-the-Practice and Future Work

This project validated the use of state-of-the-art concrete with seawater and FRP bars as internal reinforcement. The use of the state-of-the-art materials promoted sustainability by reducing the use of freshwater and increased durability of the structure by eliminating corrosion of steel reinforcement.

The use of seawater in the concrete mix yielded similar strength as the conventional concrete mix over a period of a year under laboratory conditions. To investigate the durability of concrete mix with seawater exposed to the marine environment, concrete cylinders exposed to seawater under accelerated conditions (60°C) from both freshwater and seawater mixes, will be tested for compressive strength after a year, a year and a half and two years. In addition, the extraction of concrete cores from the *iDock* after at least 10 years in service from both concrete mixes is proposed to compare the in-service durability of concrete with seawater and conventional concrete.

The use of GFRP bars as an internal reinforcement in bridge decks with over 15 years of service has been investigated and proven to be durable (Benzecry et al. 2019). The use of BFRP as an internal reinforcement, however, has not been widely implemented in in-service structures due to the lack of specifications and guidelines. The *iDock*, therefore, is the first coastal structure to implement and demonstrate the use of BFRP and, consequently, provides an opportunity for future in-service durability tests of BFRP bars.

The experimental application of non-prestressed RC-GFRP piles led to the manufacturing of two extra piles for contingency. These two extra piles were not needed in the construction of the *iDock*, and therefore, are now part of a future work. To investigate the drivability of the non-prestressed RC-GFRP piles, these two piles will be monitored while being driven in a marine environment with soil conditions typical for Miami, FL.

The implementation of bridge specifications, including AASHTO (2009, 2017, 2018) and ABC techniques proved to be a valid approach in the design and construction of this marine structure. The onsite construction of the dock lasted 13 days without any major issues, which demonstrates the advantages in construction speed and safety possible through implementation of ABC. This could allow for the design and construction method used in the *iDock* to be implemented in other marine structures.

Following the validation of materials and design concepts for the *iDock*, future work will also focus on validating the capacity of the dock to withstand a Category 5 hurricane. To investigate environmental loads associated with the dock, instrumentation will be implemented on the structure to measure wind and wave pressures under different environmental conditions. The recorded values will be used in the wind and wave simulator at the University of Miami to calibrate and test a small-scale model of the dock. The model will be tested for wind and wave loads for up to a Category 5 hurricane. Consequently, it will be possible to draw conclusions on the resilience of the structure.

Summary and Conclusions

This paper discusses the design of a marine dock that uses concrete mixed with seawater and state-of-the-practice FRP bars as internal reinforcement with positive implications in terms of durability, resilience, and sustainability. The structure served as a field demonstration for a number of innovative features and concepts:

- The structure was designed and constructed using ABC principles and techniques that are common in bridge engineering when using traditional construction materials, but not typically applied in coastal structures or coupled with the use of innovative constituent materials.

- The structure is one of the first to be designed and built according to the provisions of the second edition of *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridges* (AASHTO 2018) that was under development at the time of design (Rossini et al. 2019b). The structure was designed as a pedestrian bridge to allow for future upscaling and reuse of the technology in similar structures.
- The structure implements BFRP reinforcement in addition to more traditional GFRP reinforcement. While waiting for specific documents to be developed (Rossini et al. 2019d), the success of the project adds to archival literature supporting the extension of guidelines and specifications originally intended for GFRP reinforcement to BFRP bars.
- The structure incorporates concrete mixed with seawater in addition to traditional concrete mixed with freshwater. The mechanical performance of the two mixes were comparable and durability studies are scheduled. Freshwater is a scarce resource in some areas of the world and the use of seawater is expected to leverage sustainability over the life cycle of a concrete structure (Cadenazzi et al. 2019, 2020).
- The structure is the first of its kind to be entirely reinforced with FRP materials and is expected to feature a 100-year service life in a highly corrosive environment prone to severe hurricane events.

Data Availability Statement

All data, models, and code generated or used during the study appear in the published article.

Acknowledgments

The authors gratefully acknowledge the financial support from (1) HuRRI-Composites: Resilient Coastal Communities Using Advanced Construction Materials and Systems, Hurricane Resilience Research Institute; (2) Industry/University Cooperative Research Center, Center for the Integration of Composites into Infrastructure, National Science Foundation 1439543; and (3) Université de Sherbrooke as a research partner in this project.

Project credits go to Alan Sirkin and Alicia Sirkin, the owners; Dock and Marine Construction, the general contractor; Antonio Nanni, the engineer of record; Coreslab Structures, the precaster; Pultrall Inc., the GFRP manufacturer for the reinforcement in the piles; Owens Corning, the GFRP manufacturer for the reinforcement in the caps; Galen Composites, the BFRP bar manufacturer for the reinforcement in the slabs; Strongwell, the GFRP manufacturer for the grating; and Titan America, the concrete supplier.

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