CUMMINS | CEDERBERG
Coastal & Marine Engineering

Shoreline Inspection Report

FICA Shoreline Assessment

Fisher Island,
Miami-Dade County, Florida

August 2023

Prepared for:
Fisher Island Community Association
One Fisher Island Drive
Miami Beach, Florida 33109

Prepared by:
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Jason Taylor, PE

Geoff M. Parker, PE

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TABLE OF CONTENTS

APPENDICES

List of Photos

List of Figures

List of Tables

Shoreline Inspection Report

Fisher Island, FL

August 2023

1 INTRODUCTION

1.1 General

Cummins Cederberg, Inc. (Cummins Cederberg) was engaged by Fisher Island Community Association (FICA) to perform marine engineering and inspection services in support of managing the shoreline surrounding the island and protecting vulnerable infrastructure (Project). Cummins Cederberg is currently working with FICA on several projects, including upgrades and repairs to the ferry terminals, relocation of a dune walk-over, nourishment of the main beach and club beach shorelines, as well as with Fisher Island Club (FIC) on the improvements to the Resident's Marina. Based on Cummins Cederberg's recent observations made during engineering assessments of the failed bulkhead section within the Resident Marina, Cummins Cederberg recommended conducting an updated island-wide engineering assessment of the bulkheads and shoreline protection surrounding Fisher Island. The report presented herein summarizes the inspection results, and provides recommendations for rehabilitation, as applicable.

1.2 Scope and Objective

The Scope of Work is characterized as an updated engineering assessment of the bulkhead and shoreline protection surrounding the island to address upcoming maintenance requirements and assess funding needs over time. Specifically, the inspection included the visual assessment of the above- and below-water components of the approximate 13,000 ft of shoreline, including bulkheads, rock revetments, and rock breakwaters (ref. **Figure 1**). Certain portions of the shoreline were excluded from the Scope including the channel jetty along the south end of Government Cut Inlet (USACE jurisdiction), Resident Beach (ongoing renourishment project being performed by Cummins Cederberg), the Resident Marina (replacement marina and bulkhead currently under construction), the bulkhead fronting the TransMontaigne parcel (Folio: 30-4209-000-0040; TransMontaigne Terminals LLC jurisdiction), and the bulkhead fronting the

Resident Ferry Terminal (Alpha 21, replacement bulkhead and terminal elements under construction).

The report presented herein was developed to document the existing conditions, provide an estimate for remaining service life, and provide guidance for improvements. Service life is defined as the amount of time a structure performs adequately under its environmental and design loads. Based on the results of the inspection, recommendations for rehabilitation are provided, as applicable.

1.3 Project Location

Fisher Island, FL 33109 is located at 25.7609° N, 80.1400° W. The site is subject to average tidal variation of approximately 2 ft, with seasonal higher tides (referenced tidal station NOAA tide station ID: 8723214). The shoreline receives moderate to major wave action from vessel traffic and significant wave action during storm events. It should be noted that the North shore of the island experiences considerably more wave action on a daily basis, due to vessel traffic, as compared to the South shore.

1.4 Background Information

Prior to field inspection efforts, a desktop search for relevant background information on the existing waterfront structures and past shoreline improvements was performed. Cummins Cederberg reviewed MapDirect, Oculus, and the Miami-Dade County Online Records System. The client provided records of previous shoreline construction, rehabilitation, the 2013 report prepared by Olin Hydrographic Solutions, Inc. titled, *Fisher Island Coastal Structure Assessment*, as well as the 2019 report prepared by Edgewater Resources, LLC titled, *Fisher Island Coastal Structure Assessment* for review. Cummins Cederberg has had significant involvement with past construction throughout the island and many documents were available from previous efforts, including the 2019 report prepared for the Fisher Island Community Association titled, *Scour Assessment*.

The majority of Fisher Island's shoreline was developed in the early 1980's, with some sections of shoreline having been repaired since. Documents from the original construction and repair efforts were available for Sections 2, 5, 6, and 8 – 12, as follows (ref. **Figure 1**):

LEGEND

SECTION	TYPE	L.F.
1	RIPRAP REVETMENT/BREAKWATER	1205
2	RIPRAP REVETMENT/BREAKWATER	1392
3	ANCHORED STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	1350
4	ANCHORED CONCRETE SHEET PILE BULKHEAD W/ CONCRETE CAP	1030
5	603 LF ANCHORED CONC. PILE/PANEL BULKHEAD, ROCK TOE PROTECTION, & CONC. CAP / (2) 140 LF ROCK JETTIES	883
6	ANCHORED CONCRETE PILE/PANEL BULKHEAD W/ CONCRETE CAP	1550
7	ANCHORED STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	101
8	ANCHORED STEEL SHEET PILE BULKHEAD W/ CONC. CAP & BATTER PILES	1169
9	ANCHORED STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	939
10	STEEL COMBINATION PILE BULKHEAD W/ CONCRETE CAP	515
11	ANCHORED STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	1026
12	ANCHORED STEEL SHEET PILE & STEEL COMBI BULKHEAD W/ CONCRETE CAP	290
PARCEL 15	STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	84
13	ROCK REVETMENT	1644

CUMMINS | CEDERBERG Coastal & Marine Engineering **Miami** | Fort Lauderdale | Jupiter Tallahassee | Sarasota | St. Petersburg 201 Alhambra Circle, Suite 601 Coral Gables, FL 33134 T: +1 305-741-6155 F: +1 305-974-1969 www.cumminscederberg.com	FICA SHORELINE ASSESSMENT ONE FISHER ISLAND DR. MIAMI BEACH, FLORIDA 33109	SECTION PLAN & SHORELINE TYPE	JOB NO: 30535 DRN BY: SV SHEET: FIGURE 1	DATE: 8/24/23 CHK BY: GP

Figure 1: FICA Shoreline Inspection Plan

- The original layout of the rip rap breakwater and groin for Section 2.
- Original construction documents for Section 5 and Section 6 from 1978 and 1981, respectively.
- A 2012 rehabilitation drawing set including work completed throughout Sections 6, 8, and 9.
- Original construction documents for a length of bulkhead not replaced in Section 9, dated 2007.
- Cummins Cederberg had original shop drawings of the 2021 construction effort for Section 10.
- Limited soil profiles for Sections 8 and 11.
- 2020 Construction Drawings for Section 11
- 2019 Construction Drawings for Section 12

The historical plans for each section are referenced in Section 3.0 Structure Description, as applicable.

1.5 Stillwater Elevations

The Stillwater elevations were summarized from the effective and preliminary Flood Insurance Studies (FIS) provided by FEMA. Transect 16 was utilized for the effective FIS as this transect is located closest to the project site. The preliminary FIS had multiple transects crossing through Fisher Island; therefore, the most conservative values were provided for the Atlantic shoreline and Biscayne Bay Shoreline.

Storm Surge:

- Effective Flood Insurance Studies:
    - 100-year stillwater elevation 5.7 ft NAVD88 & 500-year stillwater elevation 6.7 ft NAVD88

- Preliminary Flood Insurance Studies:
    - Atlantic Ocean Side: 100-year stillwater elevation 6.2 ft NAVD88 & 500-year stillwater elevation 8.1 ft NAVD88
    - Biscayne Bay Side: 100-year stillwater elevation 6.5 ft NAVD88 & 500-year stillwater elevation 8.0 ft NAVD88

1.6 Scour Analysis Summary

The following is a summarization of the findings from the Cummins Cederberg 2019 report prepared for the Fisher Island Community Association titled, *Scour Assessment*. The report included an analysis of the potential scour resulting from ferries and tugboats that berth at various slips at Fisher Island including Golf Course North Wall (Section 9), North Auxiliary Slip (Section 10), and Parcel 7 (Section 11).

A scour assessment was conducted to refine previous scour calculations and estimate scour for the new commercial ferry arriving at Fisher Island. During the field investigations, observations relative to the seabed bottom and scour magnitudes were made. In-water sediment samples were collected near the locations of scour holes. A scour analysis was conducted relative to the existing vessel using the slips and observed scour conditions to obtain calibration conditions. No scour analysis was conducted for seawalls that were not being used for ferry berthing. For all areas where the scour depth is limited by the elevation of limestone, considerations should be given to potential variation in limestone elevation and verified through geotechnical analyses. The following calculated/estimated scour and limestone elevations are provided for sections relevant to the Shoreline Inspection Report:

- Golf Course North Wall (Section 9): estimated at approximately EL. -30.0 ft NAVD88 – pending geotechnical information
- North Auxiliary Slip (Section 10): updated scour elevation calculated at EL.-27.5 ft NAVD88
- Parcel 7 (Section 11): scour elevation estimated at EL. -28.0 ft NAVD88
- Limestone elevations at the toe of the walls for the areas probed varied from EL. -18 ft NAVD88 to EL. -27’ NAVD88.

The above values are only for reference and should not be used without consulting Cummins Cederberg.

2 METHOD OF INSPECTION

2.1 General

Cummins Cederberg performed the Project inspections in May (8th) and June (5th, 12th, 17th, 18th and 29th), 2023, to assess the condition of the shoreline and waterfront structures. An inspection of the existing structures was performed by an engineering dive team led by a Professional Engineer (PE) with marine structure evaluation experience, generally following the methodology presented in the American Society of Civil Engineers (ASCE) Manual on Engineering Practice No. 130: Waterfront Facilities Inspection and Assessment.

The inspection was performed as a routine assessment to document deterioration, assign condition assessment ratings, and develop recommended actions for rehabilitation. Photographs were collected during the inspection to document specific observations and are included in the body of this report. The assessment included 100% Level I (visual) and 10% Level II (cleaning) evaluation of the exposed portions of the structure (refer to **Appendix A** for ASCE Inspection Levels of Effort).

Ten locations of Level III inspection in the form of ultrasonic thickness measurements were conducted along structures comprised of steel sheet pile. The ASCE Underwater Condition Assessment Guidance is included in **Appendix B**. ASCE Element Level Condition Ratings definitions can be found in **Appendix C**. For inspection reference, marine structures can be divided into the following zones, in order of descending elevation (Ref. **Figure 2**):

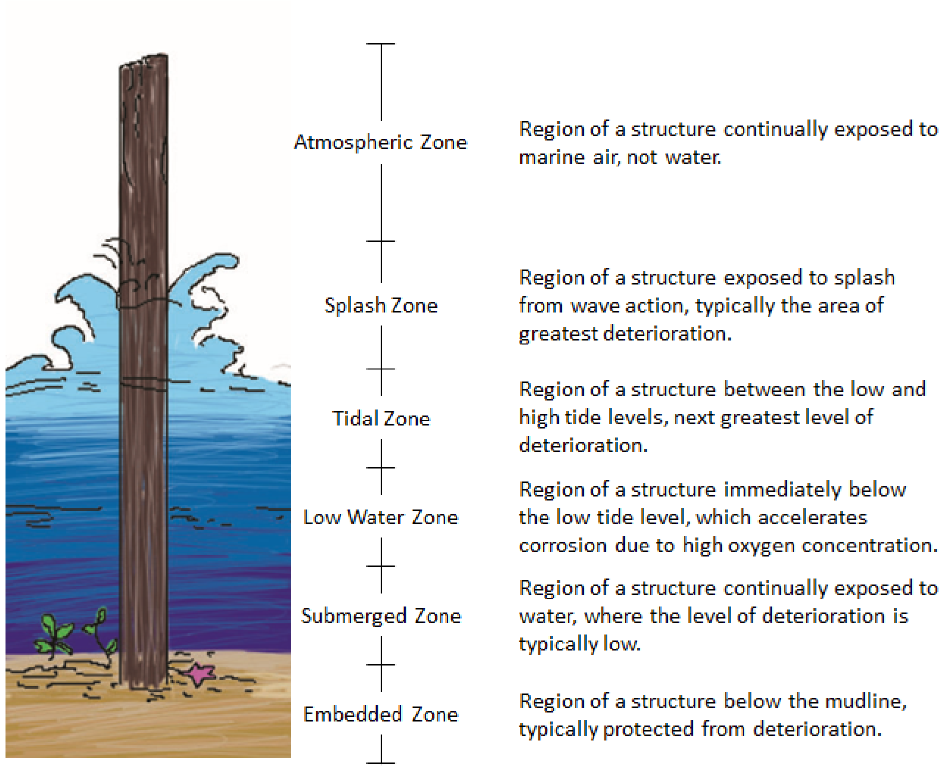


Figure 2: Deterioration Zones of Marine Structures

2.2 Procedure

Inspections were typically performed with 4-person teams. Inspections by land utilized two top-side support crew to assist with note-taking for the in-water inspection team. The in-water inspections were performed by a 2-person dive team on snorkel and/or SCUBA, including at least one Professional Engineer licensed in the state of Florida.

The bulkheads at Sections 3, 9, 12, and 13 were inspected via land deployment. The water depths enabled full in-water inspections via snorkel on Section 13. Scuba equipment was required for the investigation of Sections 3, 9, and 12.

The remaining bulkhead inspections were performed via deployment by boat. Sections 1, 2, 4, and 5 were able to be fully inspected via snorkel. The remaining sections were inspected via full SCUBA equipment.

Subject to visibility and access, structural elements were inspected at the seabed for undermining or scour. Underwater visibility varied throughout the inspections from 1-10 ft. Above- and underwater elements were inspected visually for deterioration. Upland areas were inspected for evidence of fill loss and settlement.

2.3 Signs of Deterioration

The inspection identified signs of deterioration on the waterfront structures caused by over-stressing, exposure to the marine environment, and normal service. The shoreline elements were inspected for signs of deterioration on the concrete, steel, and rock riprap or revetment components.

2.3.1 Concrete Deterioration

Concrete deterioration is generally a chemical or physical process resulting in cracking and spalling of individual members. Cracking can occur in multiple forms and from different causes, such as shrinkage cracking, subgrade settlement, and/or overloading. Cracks that develop prior to concrete hardening are referred to as plastic-shrinkage cracks or surface cracking. This occurs during the curing process of concrete in which the internal mass bleeds water to the surface and the surface water evaporates quicker than the bleed water. This differential curing produces stresses greater than the tensile strength of the concrete, causing cracking. Settlement cracking occurs when the concrete members settle or subside due to the consolidation of the supporting subgrade. Cracking can also occur when the concrete mix is over-saturated or insufficient concrete is provided over internal steel reinforcement. Proper mix design, curing methods, and reinforcement will limit the shrinkage to micro-cracking.

Width of concrete cracking is also taken into consideration to determine severity, cause of cracking, and repair options. Typically, cracks with a width up to 1/16 in. are not considered structural and may be sealed to prevent saltwater (chlorides) from entering the crack. Tidal and wave action on the bulkhead facilitates chloride intrusion into the concrete through cracks or spalls. Over time, the chlorides corrode the embedded reinforcing steel, causing further deterioration.

Concrete spalling is the delamination and loosening of concrete, typically due to corrosion of embedded reinforcing steel. The corroding steel expands and breaks the concrete bond, a process which takes place after initial surface cracking and exposure to saltwater. Closed spalls describe a partial state of delamination prior to full separation of the surface concrete from the base element. Closed spalls can be difficult to detect but are often outlined by cracking. Open spalls are visually obvious, as the surface concrete has detached from the base element, often exposing internal corroded reinforcing steel.

Concrete erosion typically occurs in the tidal zone, where there is a constant, cyclical movement of water over the surface. This cyclical action can wash away the finer aggregate of the concrete, leaving a honeycombed surface. In advanced cases, the coarse aggregate is also eroded, reducing the member section. This process is commonly referred to as ‘necking’, as it produces a distinct zone of sectional reduction in the concrete member.

2.3.2 Steel Deterioration

Corrosion is the process by which a steel component is exposed to moisture and oxygen, producing iron-oxide, or rust. Saltwater structures experience a high rate of corrosion, due to the constant exposure to chlorides in seawater or salt-laden air. The salt accelerates the corrosion process by facilitating the reaction between iron and oxygen. Resistance to corrosion can be accomplished by using corrosion-resistant steels in combination with protective or sacrificial coatings.

2.3.3 Riprap Revetment Degradation

The following damages to riprap revetment are cited in the USACE Coastal Engineering Manual (2003):

Breach/loss of crest elevation – A breach is a depression or gap in the crest of a rubble mound structure that extends to or below the bottom of the armor layer. It is caused by armor displacement. To be defined as a breach the gap must extend across the full width of the crest. Loss of crest elevation is primarily caused by settlement either of the structure or of the foundation, both of which result in a reduced structure height.

Core (or underlayer) exposure/core loss – When the underlayer or the core are clearly visible through gaps between the primary armor stones this is termed core exposure. Core loss occurs when underlayer or core is removed from the structure by waves passing through openings or gaps in the armor layer. Movement and separation of the armor stones often result in the exposure of the underlayer or core material.

Armor stone displacement – Displacement is most likely to occur near the Stillwater line, where dynamic wave and uplift forces are greatest. Localized loss of armor stone (up to four or five stones in length) is typically a pocket in the armor layer at the waterline, where the displaced stones have moved downslope to the toe of the structure.

Armor stone settling – Settling may take place along or transverse to the slope. Causes include consolidation or settlement of the underlayer stone, the core, or foundation soils.

Armor stone bridging – Bridging is a form of armor stone loss that may apply to the side slopes or the crest of a rubble mound structure. It occurs when the underlaying layers settle but the top armor layer remains in position, at or near its original elevation. This leaves a bridge over the resulting cavity, much like an arch.

Loss of armor stone contact or interlock – Armor stone contact is the edge-to-edge, edge-to-surface, or surface-to-surface contact between adjacent armor units, particularly large armor stones. Interlock refers to physical containment by adjacent armor units. Certain types of concrete armor units are designed to permit part of one unit to nest with its neighbors. In this arrangement, one or more additional units would have to move significantly to free any given unit from the matrix. Any special armor unit placement should be stated in the inspection notes.

Armor stone rounding – Rounding of armor stones, riprap, or concrete armor units with angular edges is caused by cyclic small movements or by abrasion that wears edges into smoother, rounded contours. This reduces the overall stability of the armor layer by decreasing the effectiveness of edge-to-edge or edge-to-surface contact between units and making it easier for them to move.

Armor stone spalling – Spalling is the loss of material from the surface of the armor unit. Spalling can be caused by mechanical impacts between units, stress concentrations at edges or points of armor units, deterioration of both rock and concrete by chemical reactions in seawater, freeze-thaw cycles, ice abrasion, or other causes.

Armor stone cracking – Cracking is defined by visible fractures in the surface of either armor stones or concrete armor units. Cracks may be superficial or may penetrate deep into the body of the armor unit. Cracking is potentially most serious in slender concrete armor units.

Armor stone fracturing – Fracturing occurs where cracks progress to the stage that the armor unit breaks into at least two major pieces. Fracturing has serious consequences for armor layer stability and brings a risk of imminent and catastrophic failure.

Slope defects – When loss of armor units or settlement occurs over a large enough area to change the shape or angle of the side slope this constitutes a slope defect. Slope defects occur when many adjacent armor units (or underlayer stones) appear to settle or slide as if they are a single mass. There are two forms of slope defect:

- Slope steepening is a localized process where the sloping surface appears steeper than originally designed or constructed. Steepening is evidence of a failure in progress on the slope of a rubble mound structure.

- Sliding is a general loss of the armor layer directly down the slope. Unlike slope steepening, this problem is usually caused by more serious failures at the toe of the structure. Slope failure can be caused by severe toe scour, such as can occur at a tidal inlet with strong currents, or by failure within weak, cohesive soils when soil shear strength is exceeded.

3 STRUCTURE DESCRIPTION

The Scope of Work encompassed approximately 13,000 ft of waterfront structures and shoreline protection along Fisher Island's shoreline. Stationing was established locally with respect to each section. Refer to **Figure 1** for a plan view detailing all sections inspected throughout the island.

Examination of original construction and repair documents show existing top of cap elevations in Sections 3 through 10 at approximately +4.4 ft to +5.0 ft NAVD88 excluding the stem wall step up in Section 8 to +8.4 ft NAVD88. Sections 11 and 12 are of newer construction with existing top of cap elevations at approximately +7.5 ft to +8.0 ft NAVD88.

Table 1 below describes the existing construction of each shoreline section. For reference, a combination wall (combi-wall) is a mixture of steel sheet piling (SSP) with stiffening steel elements – in this case, HZ piles. These supplementary piles have a large I- or H-shaped section that is considerably stronger and stiffer than a sheet pile.

Table 1: Existing Section Details

Section	Approximate Length	Shoreline Type
Section 1	1,205 ft	Rock Revetment / Breakwater
Section 2	1,392 ft	Rock Revetment / Breakwater
Section 3	1,350 ft	Anchored SSP Bulkhead w/ Concrete Cap
Section 4	1,030 ft	Anchored Concrete Sheet Pile Bulkhead w/ Concrete Cap
Section 5	883 ft	503 LF Anchored Concrete Pile/Panel Bulkheads w/ Concrete Cap, (2) 140 LF Rock Jetties
Section 6	1,550 ft	Anchored Concrete Pile/Panel Bulkhead w/ Concrete Cap
Section 7	101 ft	Anchored SSP Bulkhead w/ Concrete Cap
Section 8	1,169 ft	Anchored SSP Bulkhead w/ Concrete Cap & Batter Piles
Section 9	939 ft	Anchored SSP Bulkhead w/ Concrete Cap
Section 10	515 ft	Steel Combination Bulkhead w/ Concrete Cap
Section 11	960 ft	Anchored SSP Bulkhead w/ Concrete Cap
Section 12	290 ft	Anchored SSP and Steel Combination Bulkhead w/ Concrete Cap
Parcel 15	84 ft	SSP Bulkhead w/ Concrete Cap
Section 13	1,644 ft	Rock Revetment

3.1 Section 1

Section 1 is comprised of a riprap shoreline covering the southern extent of the island for approximately 890 linear ft which extends into a groin to the southeast for approximately 260 ft. An approximately 55 linear ft groin extends to the north at approximately STA. 8+90 (**Figure 3, Photo 1**). The riprap within this section was typically comprised of stable stacked stones ranging from 3-5 ft in diameter. (**Photo 2**). The riprap typically sloped at an estimated 2:1 grade into the water with deviations listed in the observations section.

Figure 3: Section 1 Stationing Plan

Photo 1: General view of Section 1 breakwater, looking southeast

Photo 2: Typical 2-6 ft diameter rip rap along Section 1

3.2 Section 2

Section 2 is comprised of a total of 1,386 linear ft of riprap. The riprap in this section forms an approximately 1,216 linear ft breakwater surrounding the southern perimeter of the marina as well as an approximately 170 linear ft groin located east of the marina and extending southeast in the offshore direction for beach protection (**Figure 4**). The riprap size ranges between 2-4 ft in diameter. The riprap typically sloped at an estimated 2:1 grade into the water.

Figure 4: Section 2 Stationing Plan

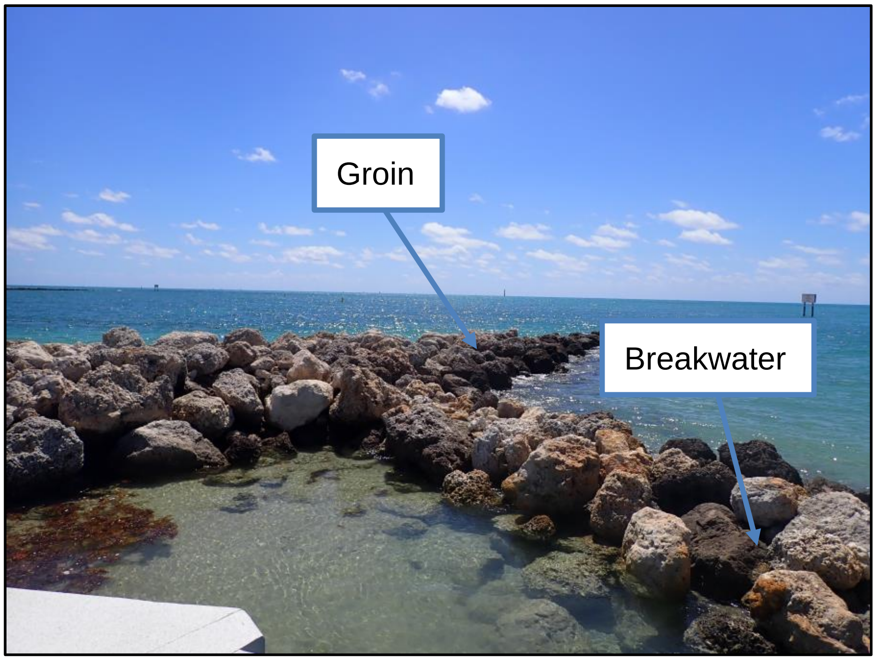


Photo 3: Section 2 groin that separates beach from boat piers, looking southeast

Photo 4: Section 2 breakwater along offshore side of Guest Marina, looking east

3.3 Section 3

Section 3 is comprised of the bulkhead that borders the Guest Marina. This bulkhead runs a total of 1,350 linear ft along the marina and includes two basins as shown in **Figure 5**.

Section 3 water depths ranged from 10 ft within the basins to 20 ft within the main portion of the marina.

Figure 5: Section 3 basin locations and Stationing Plan

The bulkhead is constructed of steel sheet piles with a concrete cap and an upland tie-back system. Hex bolts were installed on the inner flanges and are assumed to connect the upland tie rod system. Original construction documents were not found during the historical records search, but the bulkhead was shown to exist in a 1988 construction set detailing marina construction efforts. This puts the minimum age of the bulkhead at the time of inspection at 35 years, but it could be significantly older.

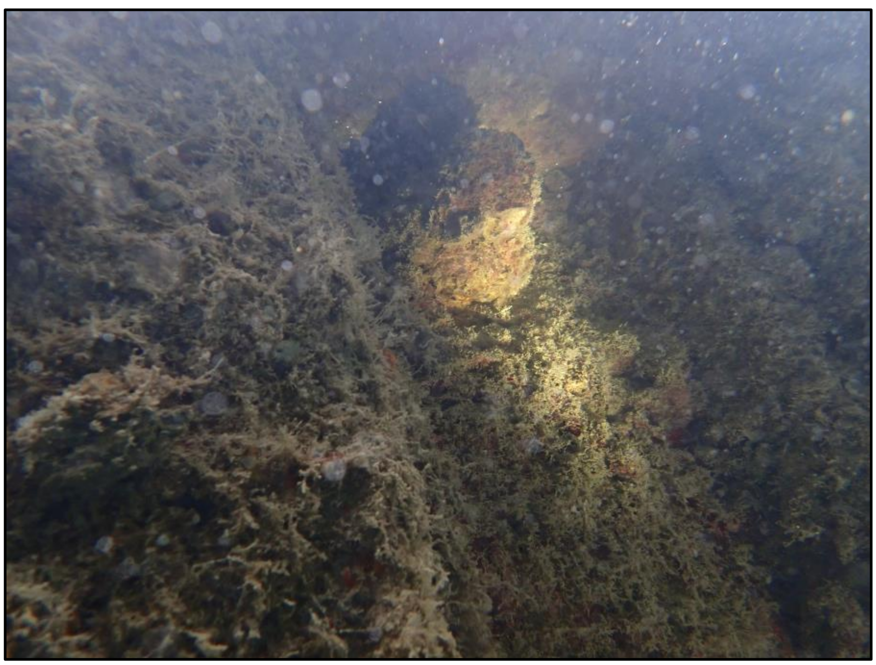


Photo 5: Typical view of anchor head on steel sheet pile at Section 3

From STA 0+00 to approximately STA 2+60, the cap measured approximately 24 in. wide. At STA 2+60, the cap extends an additional 18 in. inshore, for an approximately 42-in. wide cap. This continues until STA 3+20, where the offshore edge steps in approximately 18 in. to return to a 24 in. wide cap. The cap width is constant at 24 in. for the remainder of the section.

Photo 6: Section 3 increase in cap width at approximate STA 2+60

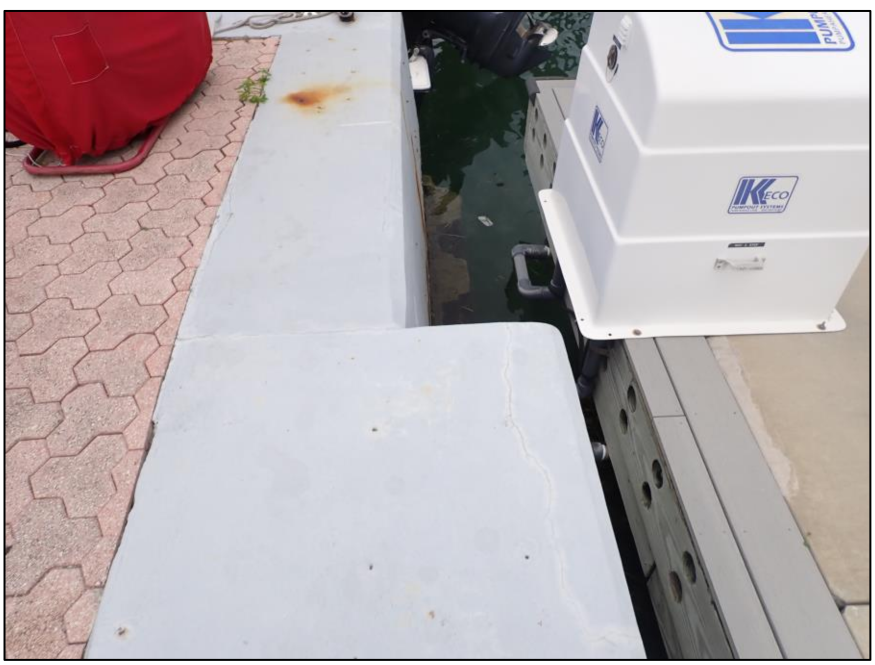


Photo 7: Section 3 reduction in cap width at offshore face, at approximate STA 3+20

At the location where the cap steps inshore, the bulkhead remains in the same plane, exposing the top of the sheet piling. The ribs of the sheet piling have been grouted outside of the cap.

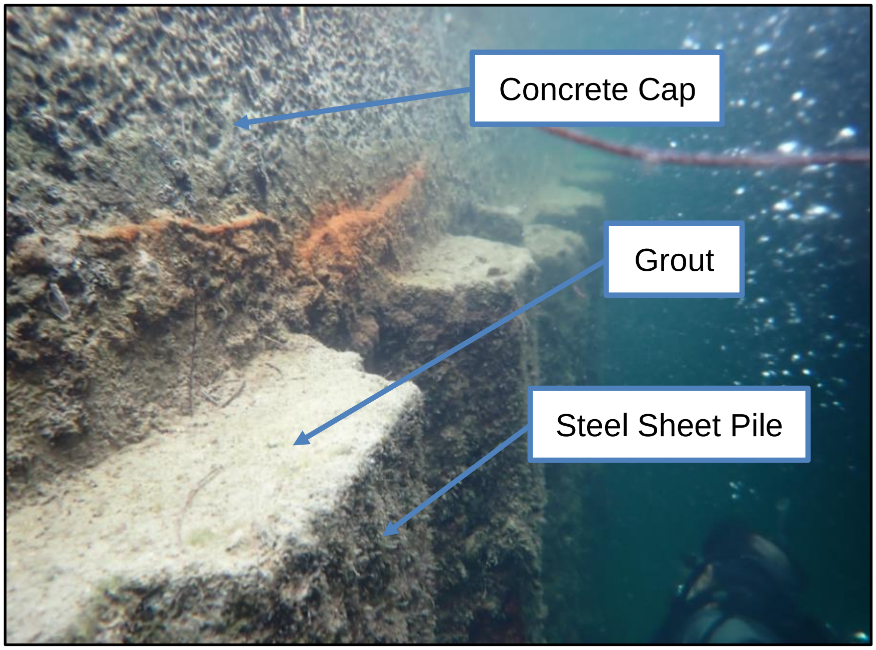


Photo 8: Typical Section 3 bulkhead construction from STA 3+20 to the end

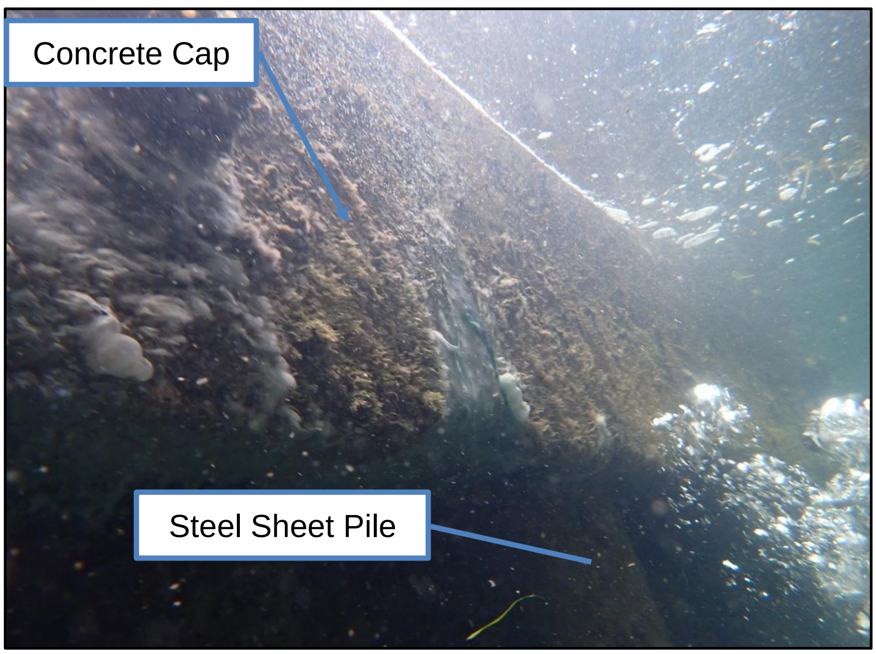


Photo 9: Typical Section 3 bulkhead construction from STA 0+00 to STA 3+20

3.4 Section 4

The bulkhead at Section 4 begins at the Guest Marina boat ramp and extends to the Resident Marina entrance. It is comprised of approximately 1,030 linear feet of concrete sheet pile bulkhead with a concrete cap and an upland tie-back system (**Figure 6**). The tongue and groove concrete sheet pile measured approximately 4-ft wide. Original construction plans were not found so the date of construction and lateral support design are unknown. Section 4 water depths ranged from 0 ft at the boat ramp to 10 ft along the length of the bulkhead.

Figure 6: Section 4 Stationing Plan

Photo 10: Eastern termination of Section 4 concrete sheet pile showing tongue-and-groove construction

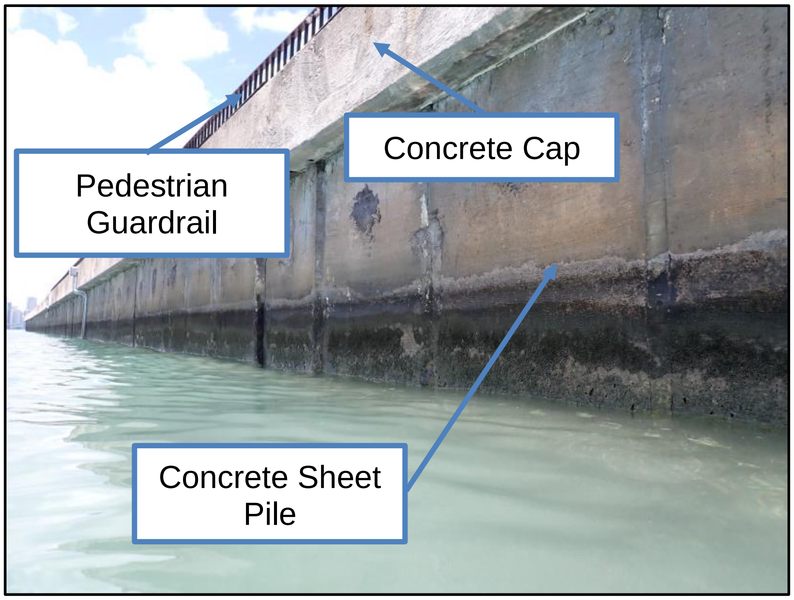


Photo 11: Typical view of Section 4 bulkhead construction

The concrete cap measured 24-in. wide and 16-in. deep with an 8-in. soffit overhang beyond the concrete panels. Two 14-in. diameter corrugated metal outfalls were present above the waterline at STA 0+25 and STA 1+52. One 10-in. diameter corrugated metal outfall was present below the waterline at STA 4+09.

Photo 12: Typical corrugated metal outfalls at Section 4: 14 in. at STA 1+52 (left) and 10 in. at STA 4+09 (right)

3.5 Section 5

Section 5 is comprised of the shoreline along the entrance channel to the Resident Marina. This includes approximately 311 LF of the eastern bulkhead and 292 LF of the western bulkhead, along with the riprap fronting the bulkhead. There are also rock jetties at the ends of the channel, each approximately 140 LF (**Figure 7**). Section 5 water depths ranged from 0 ft at the toe of the riprap to approximately 10 ft at the southern terminations of the bulkheads. Stationing for the bulkheads was taken at the southern extent for the eastern bulkhead and was restarted at the northern extent for the western bulkhead.

Figure 7: Section 5 Stationing Plan

The bulkhead is comprised of concrete king pile and panel construction with a concrete cap and an upland tie-back system. The king piles were 12-in. square precast concrete. The cap measured approximately 24-in. wide and 18-in. deep, with a 16-in. soffit overhang from the panels. Documents from the original construction were obtained, dated 1978. The original design section can be seen in **Figure 8**. During the upland tie rod investigation, the rods were observed at approximately 3-4 ft below the top of cap, a significantly lower elevation than shown in the original design drawings.

Riprap was present along the full extent of the inspection scope and typically sloped from the mudline to within 2 ft to 4 ft below the top of the cap. One rock jetty extends from the southwest entrance of the basin and the other from the northeast, creating a buffer for wave action to protect the vessels of the Resident Marina. The rocks typically measure 2 -4 ft in diameter along the bulkheads and 3-5 ft in diameter in the jetties.

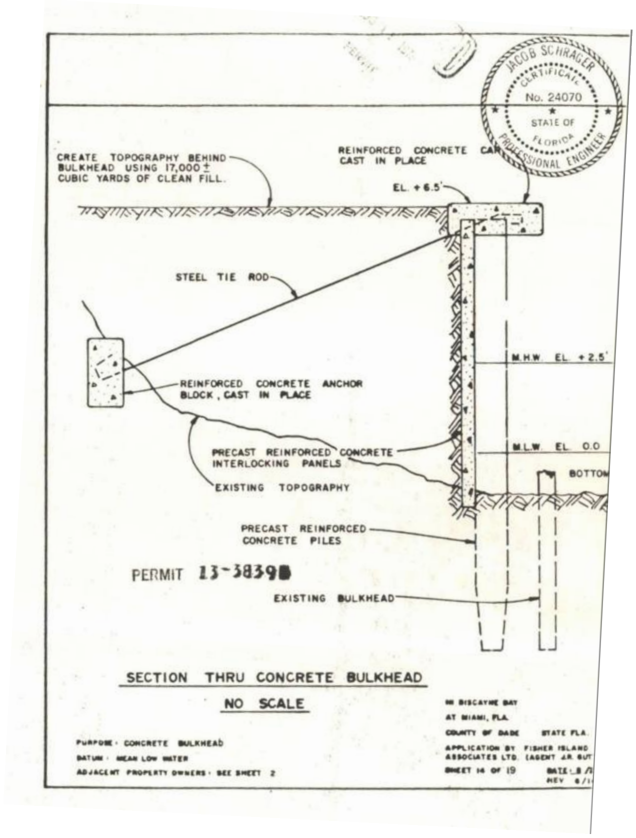


Figure 8: Original design for Section 5 bulkhead

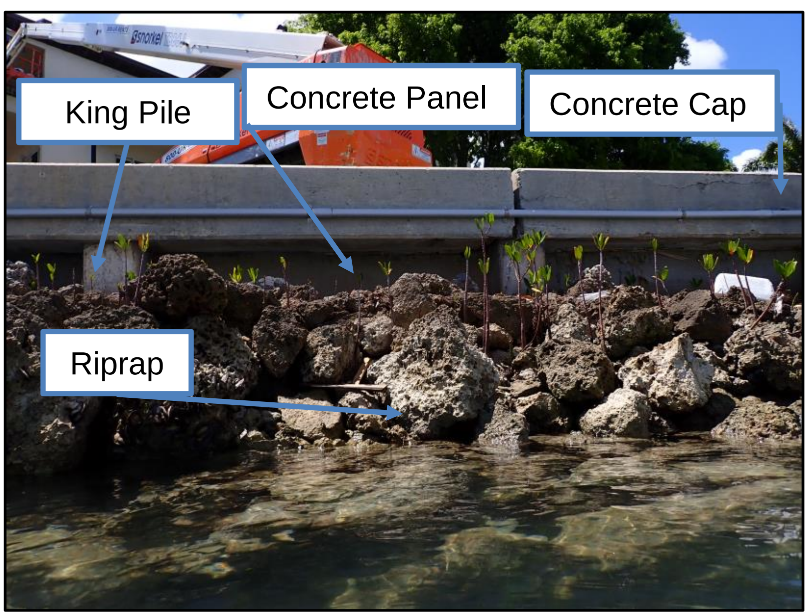


Photo 13: Typical view of Section 5 shoreline construction

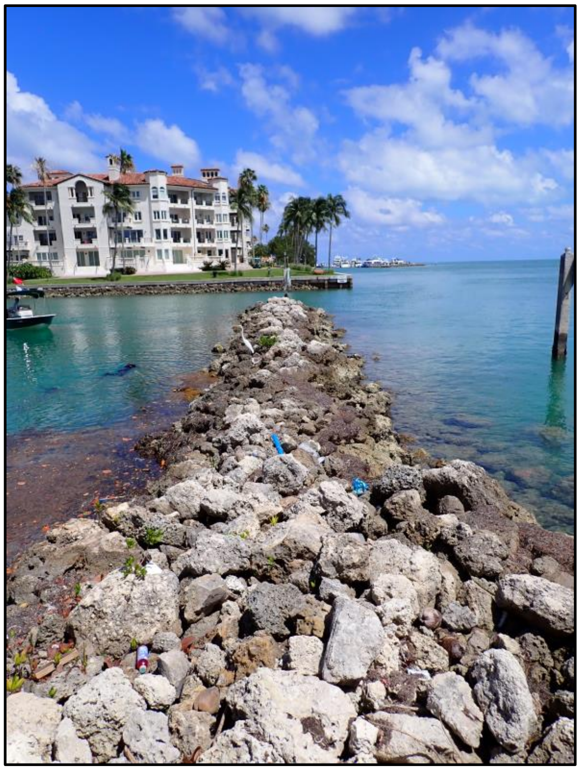


Photo 14: STA 2+80 view of Section 4 Southwest jetty at outlet of harbor

Photo 15: STA 2+60 view of Section 4 Northeast jetty at marina basin

3.6 Section 6

Section 6 is comprised of a concrete panel and king pile bulkhead, with a concrete cap and an upland tie-back system. The bulkhead begins at the mouth of the Resident Marina, extends northwest for approximately 1,550 linear ft, and terminates at Section 7 (**Figure 9**). Section 6 water depths ranged from 10 ft to 15 ft. The concrete cap was measured to be approximately 30-in. wide and 18-in. deep. At approximately STA 14+90, the cap width changes to 24-in., with the same depth. Plans from the original construction were obtained, dated 1981; the typical section can be seen in **Figure 10**. During the tie rod investigation, the rods were observed at approximately 4-5 ft below the top of cap, a significantly lower elevation than shown in the original design drawings.

Figure 9: Section 6 Stationing Plan

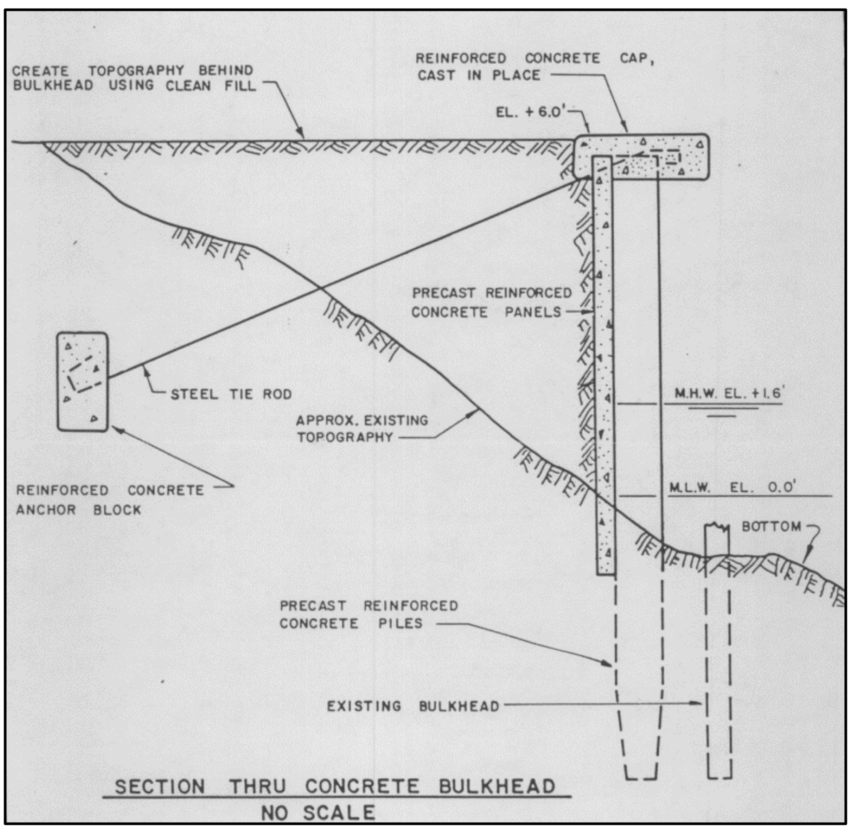


Figure 10: Original design for Section 6

Photo 16: Typical view of Section 6 bulkhead construction

The king piles are originally 12-in. square concrete piles but were encased in the 2012 bulkhead rehabilitation effort. The encasements provided 4 in. of additional concrete on each face and were measured at 16-in. deep and 20-in. across. The encasements typically extended from the cap down approximately 4-5 ft. The rehabilitation effort also included a steel sheet pile toe wall at the base of the bulkhead, riprap and grout bags at the toe of the panels, concrete patching of the king pile-panel interface, and crack repairs on the concrete cap. The rehabilitation section and pile jacket detail can be seen in Figure **11**.

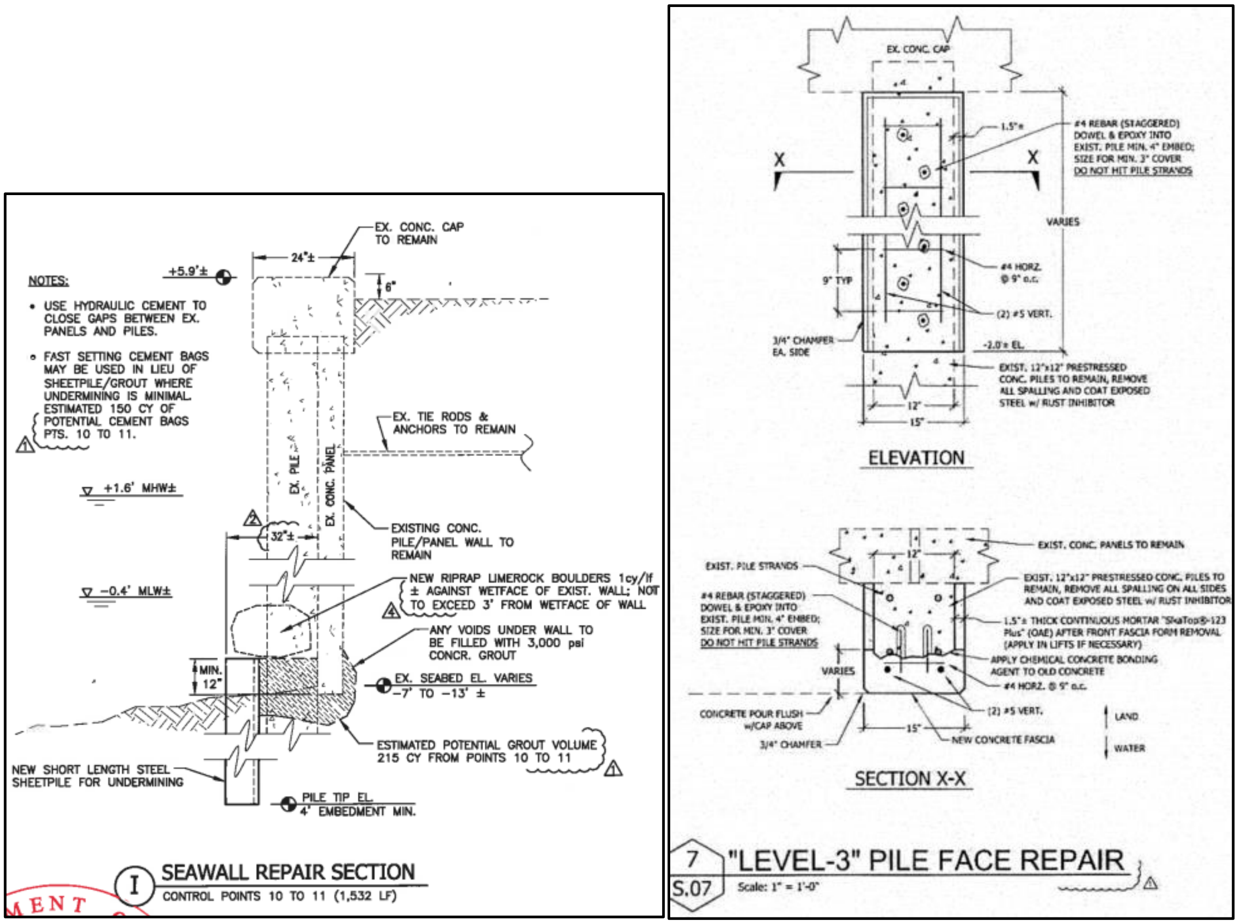


Figure 11: Section 6 bulkhead repair drawings, including pile encasement detail

Photo 17: Intermittent grout bag installation along the toe of Section 6

Photo 18: Steel sheet pile toe wall along the base of the Section 6 bulkhead

Outfalls observed included:

- a 16-in. diameter steel pipe at STA 14+75, with an installed manatee grate,
- a 16-in. diameter steel pipe at STA 18+80 protruding through a 20-in. high by 24-in. wide panel opening, with bricks infill, and
- a 15-in. diameter PVC pipe at STA 26+55.

Photo 19: Section 6 outfalls

3.7 Section 7

Section 7 begins at the Section 6 termination and runs north for approximately 101 ft, ending at Section 8 (**Figure 12**). Section 7 water depths ranged from 10 ft to 15 ft. The bulkhead is comprised of steel sheet piles with a concrete cap and an upland tie-back system. The concrete cap was measured at 30-in. width and 18-in. depth. The 2012 bulkhead rehabilitation stated that Section 7 was excluded from the scope of work. Original construction plans were not found, so the original sheet pile section, date of construction, and anchor system are not fully documented. The observed tie rods appeared to be significantly newer than the installed steel sheet pile and are reported by the client to have been part of a retrofit effort.

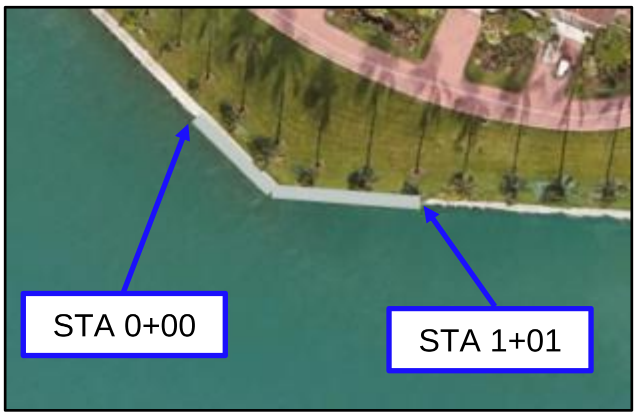


Figure 12: Section 7 Stationing Plan

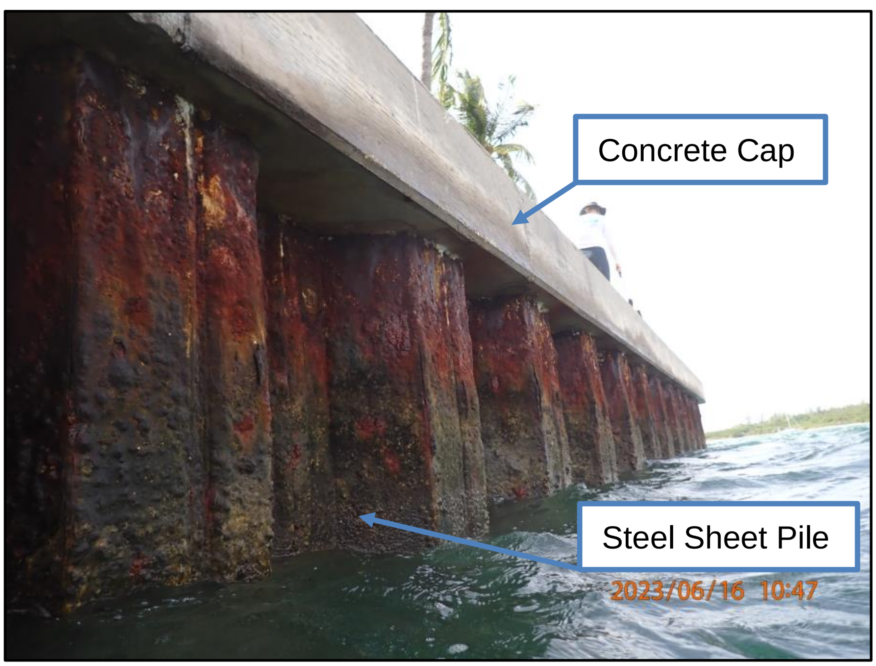


Photo 20: Typical view of Section 7 bulkhead construction

3.8 Section 8

Section 8 consists of approximately 1,169 linear ft of bulkhead with water depths ranging from 10 ft to 15 ft (**Figure 13**). Starting at the western extent of the Section 9 connection and continuing 309 LF to the west, the Section 8 bulkhead is comprised of steel sheet piles with a concrete cap and presents two battered concrete piles at every three ribs. The sheet pile was listed as the AZ28-700 section in the 2012 repair drawings. AZ28-700 has a specified flange and web thickness of 0.52 in. The average ultrasonic thickness measurement (UTM) readings during the inspection were 0.562 in for the flanges and 0.412 in for the web, with no loss of coating at the measured locations. It is likely that an equivalent section was selected during construction, based on material availability. The battered piles are comprised of an 18-in. by 18-in. prestressed concrete section and were driven to a tip elevation of -47 ft (NGVD29; based upon plans, not verified). A new 48-in wide by 24-in deep cap was installed with a 16-in thick by 47-in high stem wall structurally integrated with the cap. The stem wall steps upward incrementally for a total of 3'-11" in at the Section 9 interface and terminates at STA 3+09 (see **Photo 21 & Figure 14**).

Figure 13: Section 8 Stationing Plan

Photo 21: Stepped stem wall at interface between Sections 8 and 9

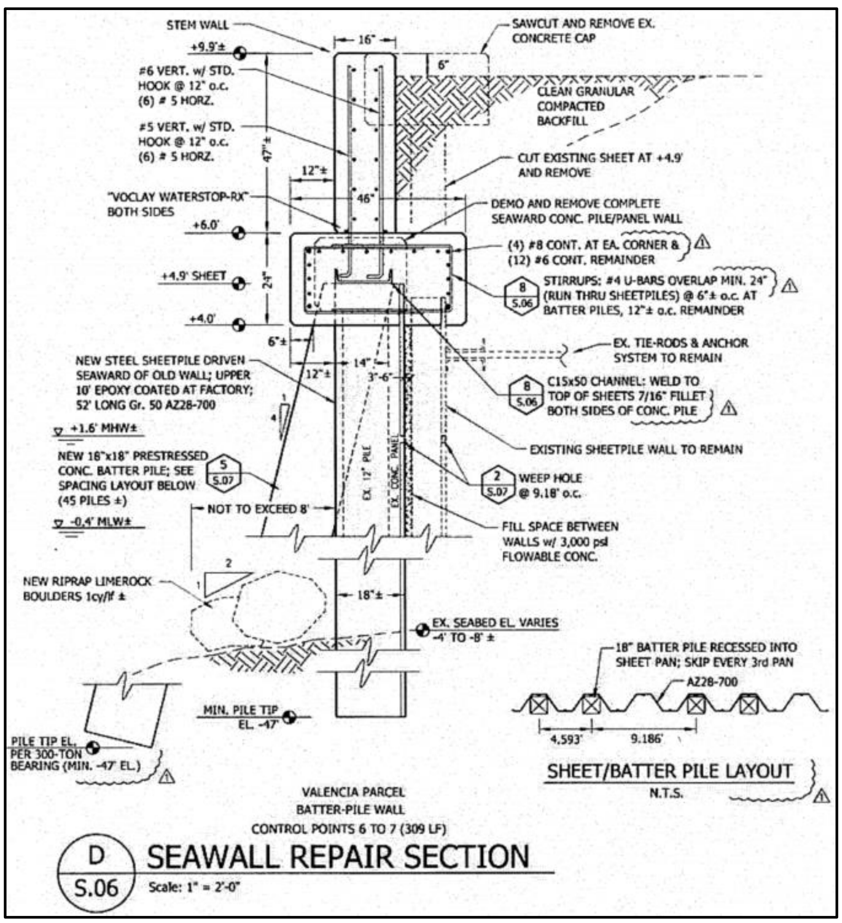


Figure 14: Drawing of 2012 Section 8 rehabilitation, STA 0+00 – STA 3+09

Photo 22: Section 8 stem wall termination and construction change at STA 3+09, looking southwest

From STA 3+09 to STA 6+64, the bulkhead is comprised of steel sheet piles with a concrete cap. The concrete cap was measured at 24-in height and 30-in width. No battered piles were present, as the bulkhead is supported by inshore ground anchors cast into the cap. These sheets were also installed during the 2012 rehabilitation effort (**Figure 15**). The sheet pile section specified is AZ26-700.

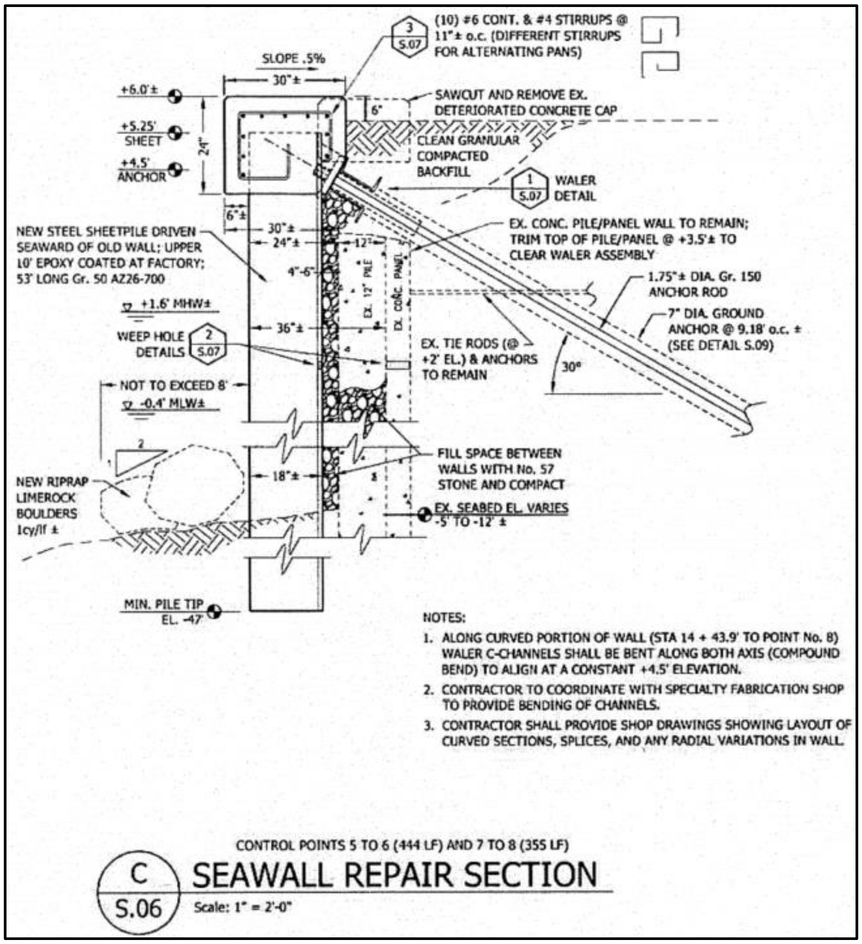


Figure 15: Section 8 reconstruction drawing, STA 3+09 – STA 6+64 (2012)

Photo 23: Typical view of Section 8 bulkhead, STA 3+09 to STA 6+64, facing west

From STA 6+64 to the termination at Section 7 (STA 11+69), the bulkhead is comprised of steel sheet piles supported by a concrete cap with prestressed concrete piles driven in two out of every three ribs (**Figure 16**). The bulkhead was installed during the 2012 rehabilitation effort and the section specified was an SCZ21 sheet. The UTM readings taken aligned with the specified sheet. The battered piles are comprised of a 14-in by 14-in prestressed concrete section and were driven to a tip elevation of −30 ft (NGVD29; based upon plans, not verified). The concrete cap measured 48-in wide by 22-in deep.

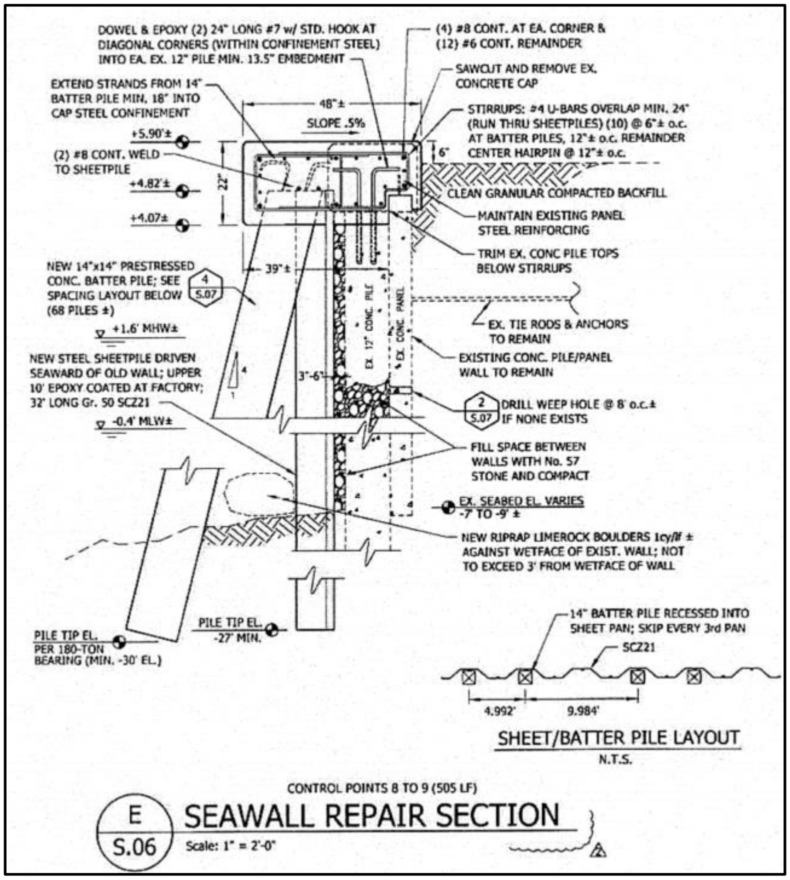


Figure 16: Section 8 reconstruction drawing, STA 6+64 – STA 11+69 (2012)

Photo 24: Typical view of Section 8 bulkhead, STA 6+64 to STA 11+69, facing north

Review of the 2013 report prepared by Olin Hydrographic Solutions, Inc. titled, *Fisher Island Coastal Structure Condition Assessment* yielded a limited soil profile as a part of a bulkhead analysis model for Section 8. The profile shows sand and silty sand to an approximate depth of 26.5 ft below existing grade where layers of limestone and sand were encountered (see **Figure 17**).

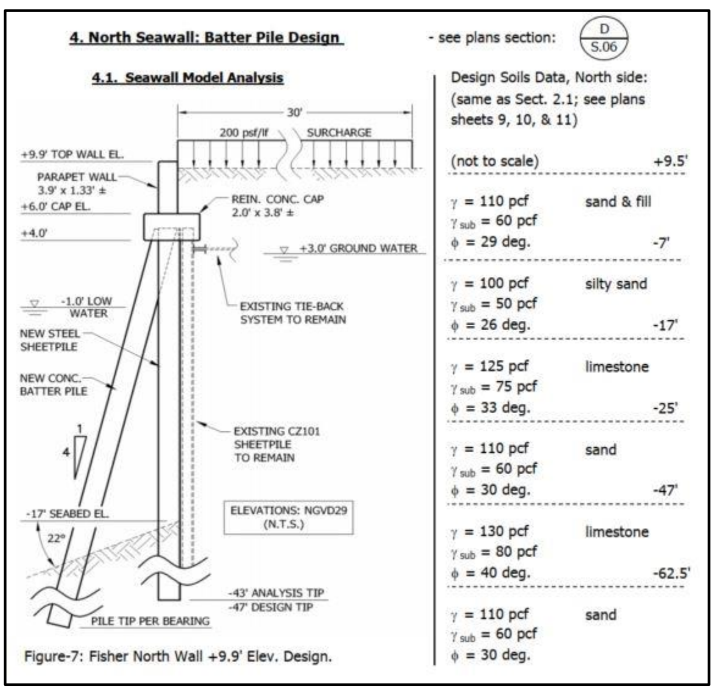


Figure 17: Section 8 limited soil profile and bulkhead analysis model

3.9 Section 9

Section 9 is comprised of a total of 939 LF of bulkhead with varying construction (**Figure 18**). The bulkhead from STA 0+00 to STA 0+30 is comprised of steel sheet piles with a concrete cap. Section 9 water depths ranged from 5 ft to 20 ft. Original construction documents for this section were not found and this section was out of scope for a 2012 rehabilitation effort. A 1999 Barge docking facility bulkhead repairs plan listed the existing steel sheet piles as a BZ-37 profile. The profile of the sheet piles observed appeared similar to what was shown in the 1999 repair set. The section is assumed to be a BZ-37 with a minimum age of 24 years at the time of inspection and likely substantially more. Additionally, the 2012 drawing set states that the scope of work began at STA 0+64; however, the sheet pile observed at STA 0+30 is consistent with the section repaired in 2012 stated to be from STA 0+64 to STA 0+85.

Figure 18: Section 9 Stationing Plan

From STA 0+30 to STA 0+85 the bulkhead is comprised of steel sheet piles with a concrete cap and ground anchors cast into the cap (**Figure 19**). The bulkhead was installed as part of the 2012 repair effort and the sheet was specified as an AZ26-700.The plans stated that the sheets were to be installed from STA 0+64 to STA 0+85 but were observed from STA 0+30 to STA 0+85.

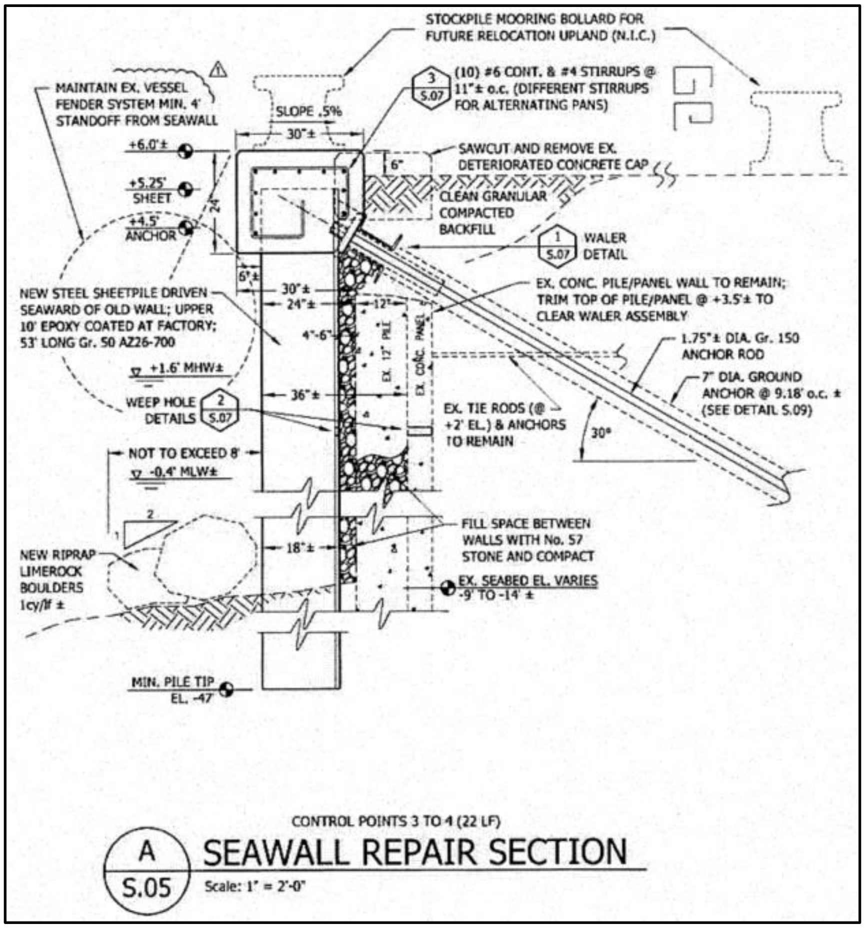


Figure 19: Section 9 reconstruction drawing, STA 0+64 – STA 0+85 (2012), observed STA 0+30 to STA 0+85

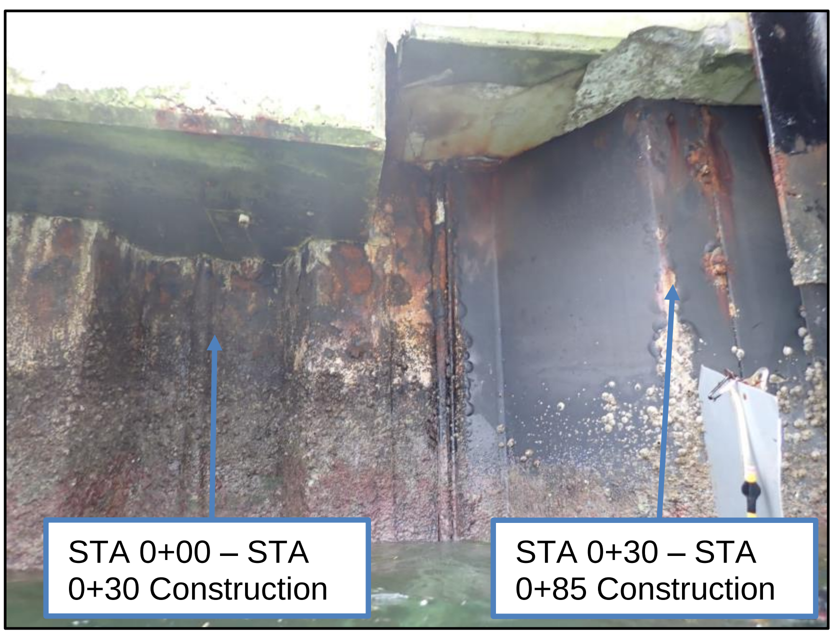


Photo 25: Section 9 construction change at STA 0+30

From STA 0+85 to STA 4+64 the bulkhead is comprised of steel sheet piles with a concrete cap and an upland tie-back system (**Figure 20**). Historical drawings show the tie rods are connected to the sheet piling with a continuous double channel steel waler. From STA 0+64 to STA 4+64 a steel sheet pile bulkhead was driven waterward of the existing to address a failed section in 2007. The sheet section was specified to be AZ38-700, or approved equal, in the general notes. UTM readings taken at this section align with the assumption of an AZ38-700 sheet. The 2012 rehabilitation effort stated that repairs were performed from STA 0+85 to STA 4+95 (**Figure 21**), but new sheet piles were not installed. A construction joint was observed at STA 4+64, which aligns with the 2007 stationing. The weep holes installed in 2012 were observed with jet filters. The weep holes facilitate drainage of the backfill behind the bulkhead, while the filters keep the backfill from eroding through the holes.

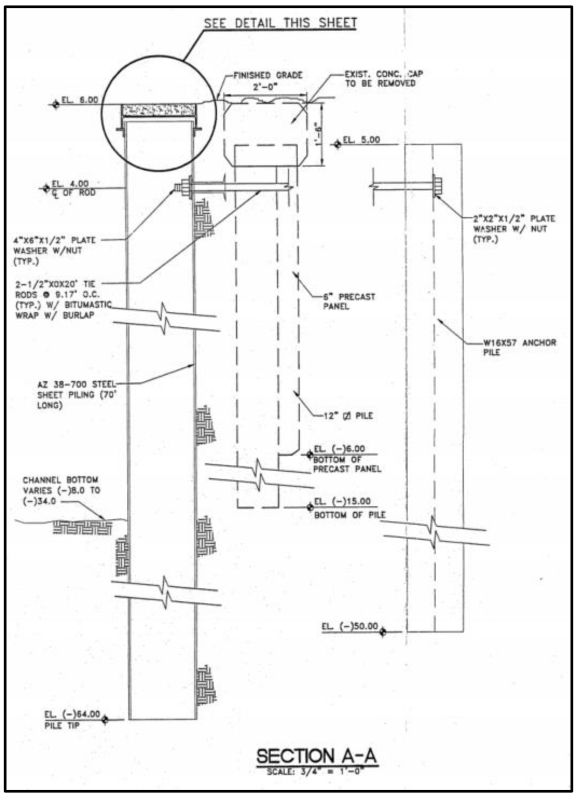


Figure 20: Section 9 2007 Bulkhead installation from STA 0+85 – STA 4+64

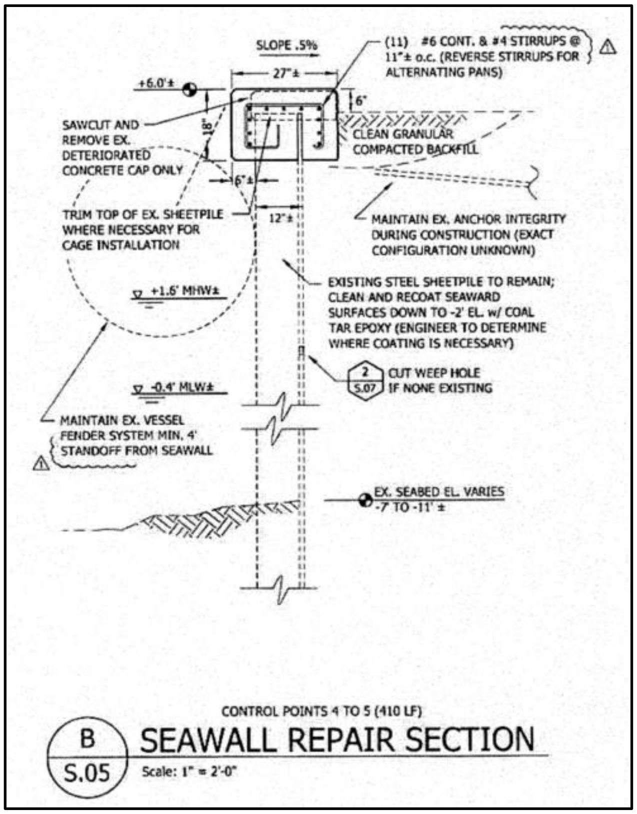


Figure 21: Section 9 reconstruction drawing, STA 0+85 – STA 4+95 (2012), observed STA 0+85 to STA 4+64

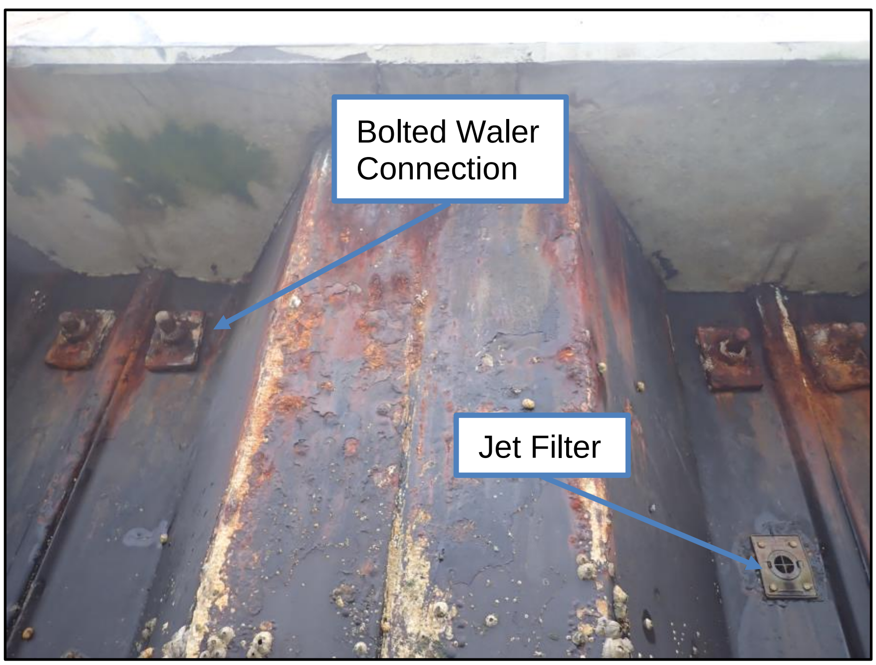


Photo 26: Section 9 typical bulkhead construction STA 0+85 – STA 4+64

From STA 4+64 to STA 9+39 (Section 8 transition) the bulkhead is comprised of steel sheet piles with a concrete cap and ground anchors cast into the cap (**Figure 22**). The 2012 rehabilitation effort stated that new sheet piles were driven from STA 4+95 to STA 9+39, but the construction joint was observed at STA 4+64. This construction matched the Section 8 construction from STA 3+09 to STA 6+64 (Section 8 Stationing). This included the AZ26-700 sheets with a 30-in by 24-in concrete cap.

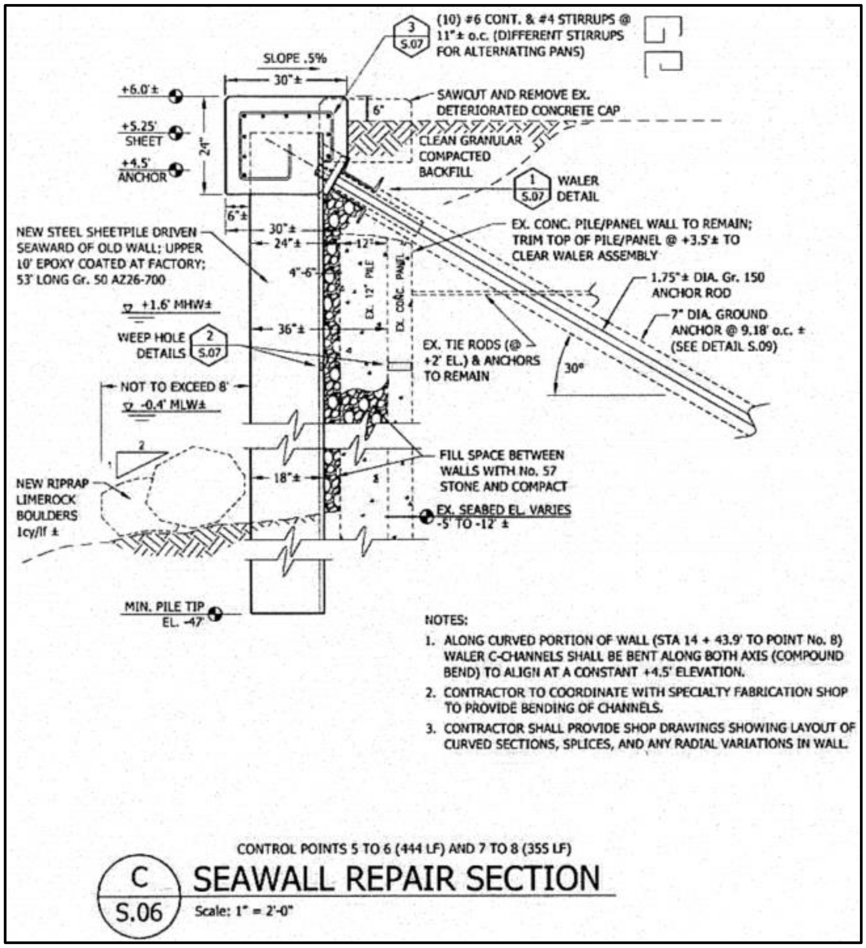


Figure 22: Section 9 reconstruction drawing, STA 4+95 – STA 9+39 (2012), observed STA 4+64 to STA 9+39

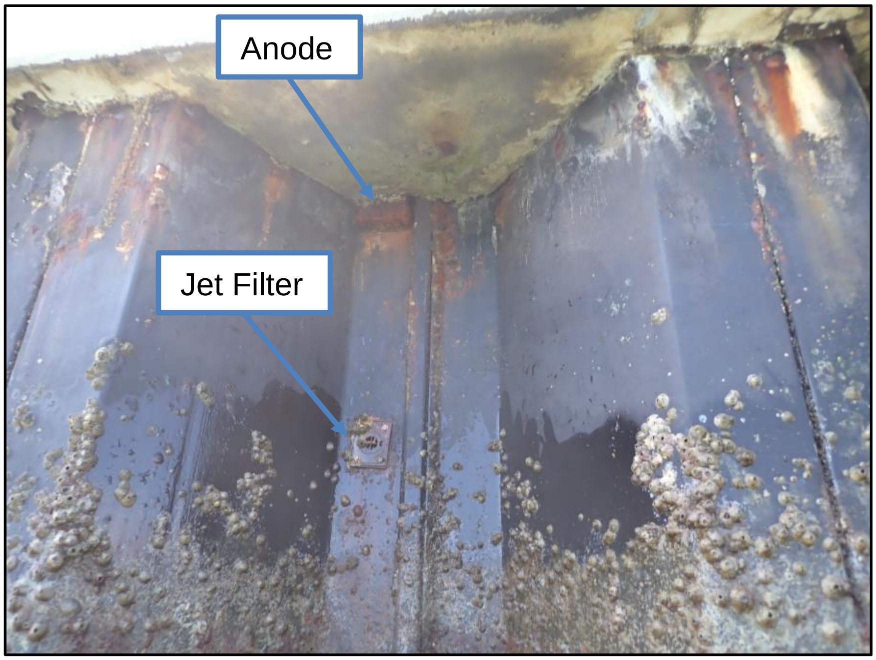


Photo 27: Section 9 typical bulkhead construction STA 4+64 – STA 9+39

3.10 Section 10

The Section 10 bulkhead is comprised of a steel combi-wall with a concrete cap. The bulkhead begins at the western termination of Section 9 and continues for approximately 470 linear ft forming a ferry basin (**Figure 23**). Section 10 water depths ranged from 10 ft to 20 ft. Shop drawings from 2021 were available stating the wall to be comprised of AZ19-700 sheet piles combined with HZ1080M king piles. The cap is 7-ft high and 4-ft wide. Fender panels are placed intermittently along the east and west basin walls (**Figure 24** & **Figure 25).**

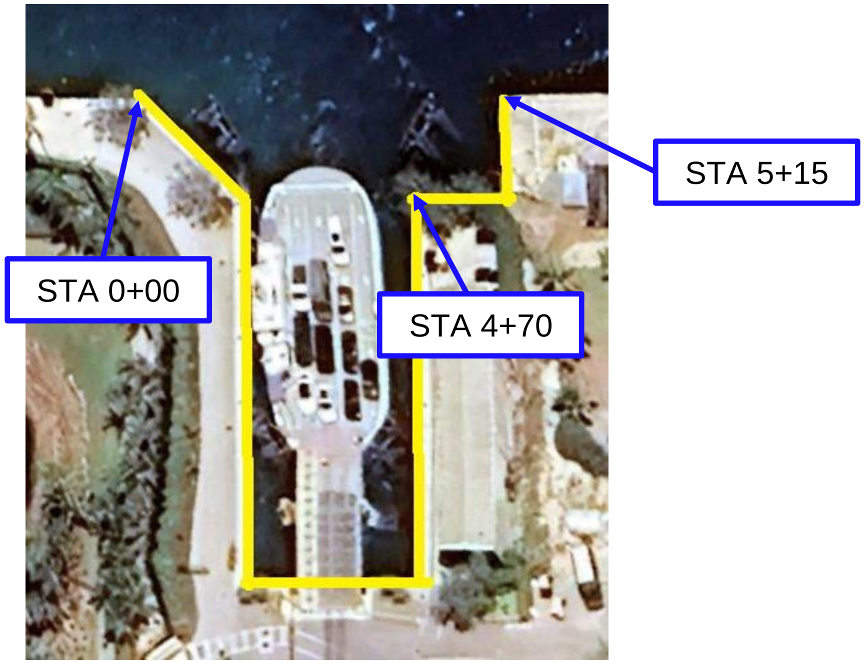


Figure 23: Section 10 Stationing Plan

Photo 28: Overall view of Section 10 ferry basin

Photo 29: Typical view of Section 10 Combi-Wall

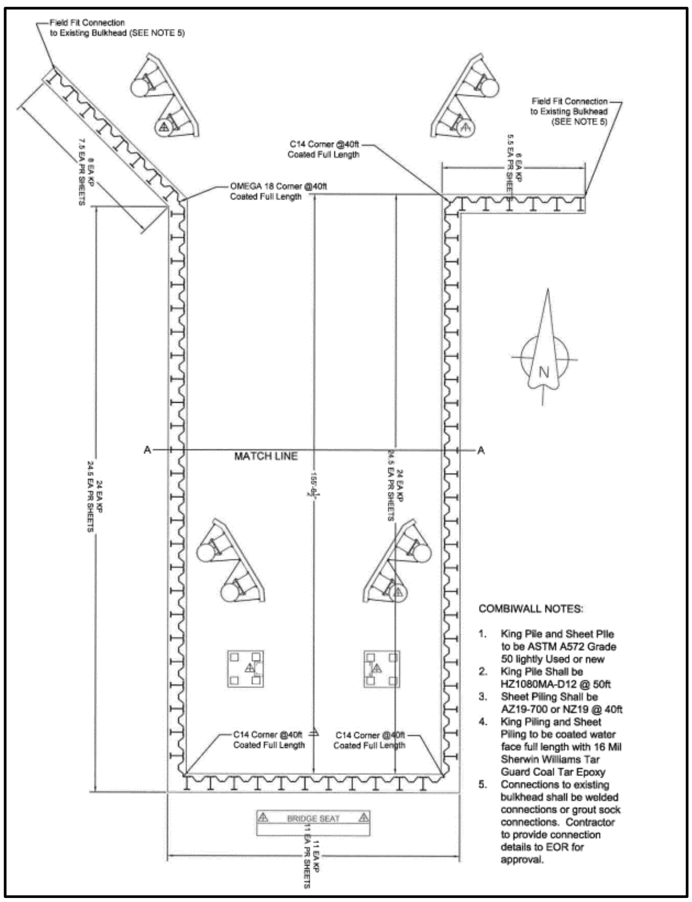


Figure 24: Historical plans for the Section 10 combi-wall

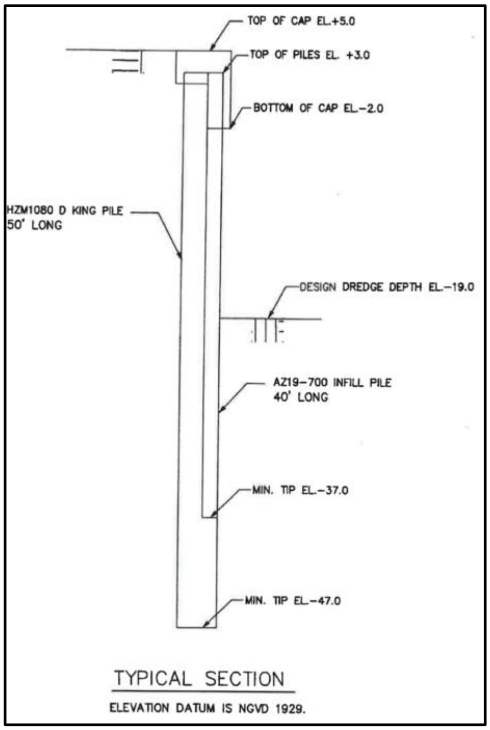


Figure 25: Bulkhead Section 10 bulkhead typical construction

The 45 linear ft bulkhead located to the northeast of the basin running north-south was included in the inspection. This length of bulkhead is comprised of aging steel sheet piles with intermittent sections of deteriorated concrete cap remaining. Original construction plans were not found; therefore, the original sheet pile section, date of construction, and lateral support design are not documented.

Photo 30: Typical view of northeast of Section 10 ferry basin bulkhead

3.11 Section 11

The Section 11 bulkhead begins at the southwest corner of the central northern ferry basin and runs approximately 1,026 linear ft to its termination at the western extent of the Resident Ferry Terminal (Alpha 21). Refer to **Figure 26** below for the Stationing Plan. The inspection commenced in the western half of the basin (Station 0+00). The original bulkhead construction within the basin is comprised of steel sheet pile with a concrete cap, which were installed in approximately 1982. A new steel sheet pile bulkhead was constructed landward of the original in approximately 2021, from STA 0+84 to STA 9+60. Section 11 water depths ranged from 10 ft to 25 ft.

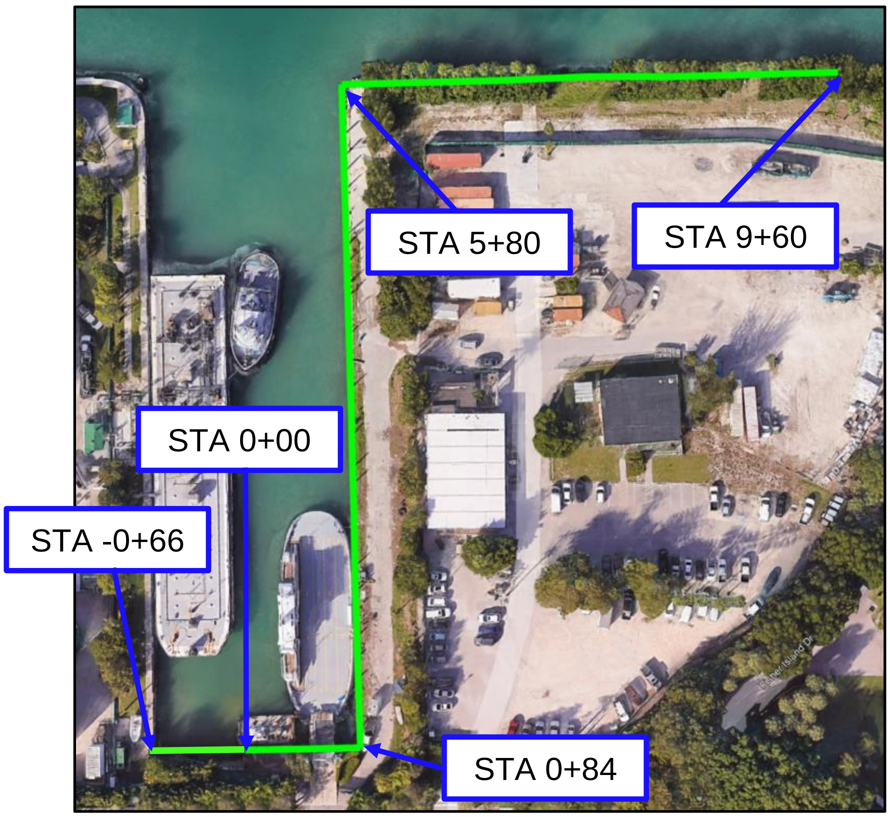


Figure 26: Section 11 Stationing Plan

Photo 31: Section 11 change in concrete cap construction at STA 0+84 interface

Photo 32: Derelict bulkhead steel sheet piles visible within Section 11 ferry basin

Photo 33: Original steel sheet piling along Section 11 northern bulkhead, STA 5+80 – 9+60

Photo 34: Typical construction of concrete cap throughout Section 11

Review of the 2013 report prepared by Olin Hydrographic Solutions, Inc. titled, *Fisher Island Coastal Structure Condition Assessment* yielded a limited soil profile as a part of a bulkhead analysis model for Section 11. The profile shows sand and silty sand to an approximate depth of 26.5 ft below grade, where layers of limestone and sand were encountered (see **Figure 27**).

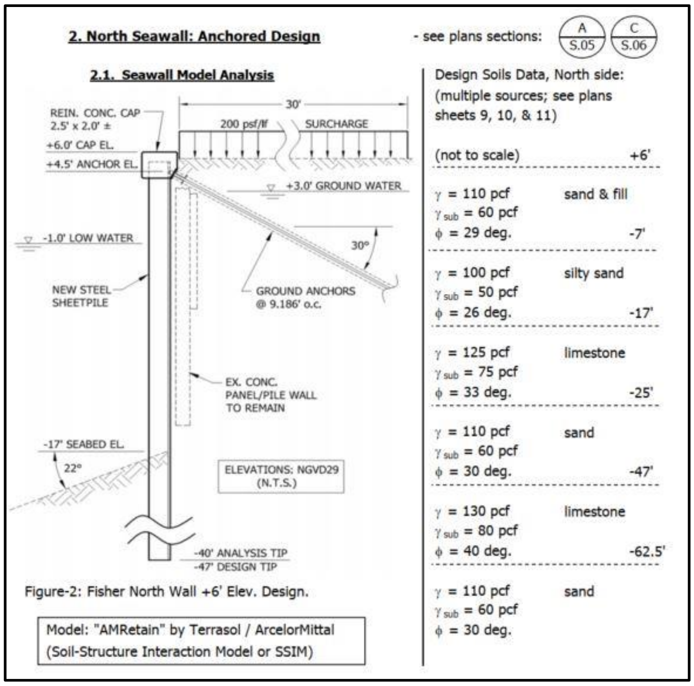


Figure 27: Section 11 limited soil profile and bulkhead analysis model

3.12 Section 12

Section 12 is comprised of approximately 290 linear ft of waterward aging steel sheet pile bulkhead a newly installed steel combi-wall landward of the original in approximately 2018. The bulkhead begins at the northwest corner of the northeast basin and terminates to the northeast within the Section 13 riprap revetment (**Figure 28**). The original concrete cap was demolished and replaced with a new concrete cap during the combi-wall construction. The cap is stepped with a 50-in wide base. The cap presents a stem wall aligned with the upland edge of the base. This stem wall measures 14-in wide and 36-in high from STA 0+50 to STA 1+50 and 24-in high throughout the rest of the cap. Section 12 water depths ranged from 5 ft to 15 ft.

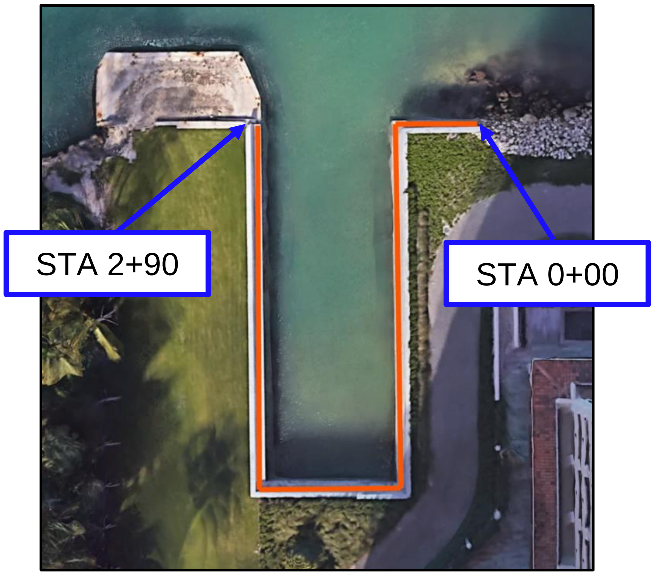


Figure 28: Section 12 Stationing Plan

Photo 35: View of Section 12 basin

Photo 36: Section 12 View of new pipe pile portion of combi-wall through deterioration in original outer sheet

Photo 37: Section 12 View of new steel sheet pile portion of combi-wall through deterioration in original outer sheet

Photo 38: Northeastern termination of Section 12 bulkhead into Task 2 riprap revetment

3.13 Parcel 15

The Parcel 15 bulkhead begins at the northwest termination of the Section 12 basin and runs approximately 84 linear ft. to its termination at the Alpha 21 ferry terminal. Refer to **Figure 29** below for the Stationing Plan. Original construction documents for the structure were not available, hence the age and original cross-section details are unknown. The bulkhead is comprised of steel sheet pile with a concrete cap throughout. The top of cap elevation is located approximately 1 ft below the Section 12 cap elevation and approximately 3 ft below the Section 12 top of stem wall elevation. Parcel 15 water depths ranged from 10 ft to 20 ft.

Figure 29: Parcel 15 Stationing Plan


Photo 39: STA 0+00 Parcel 15 termination at Section 12

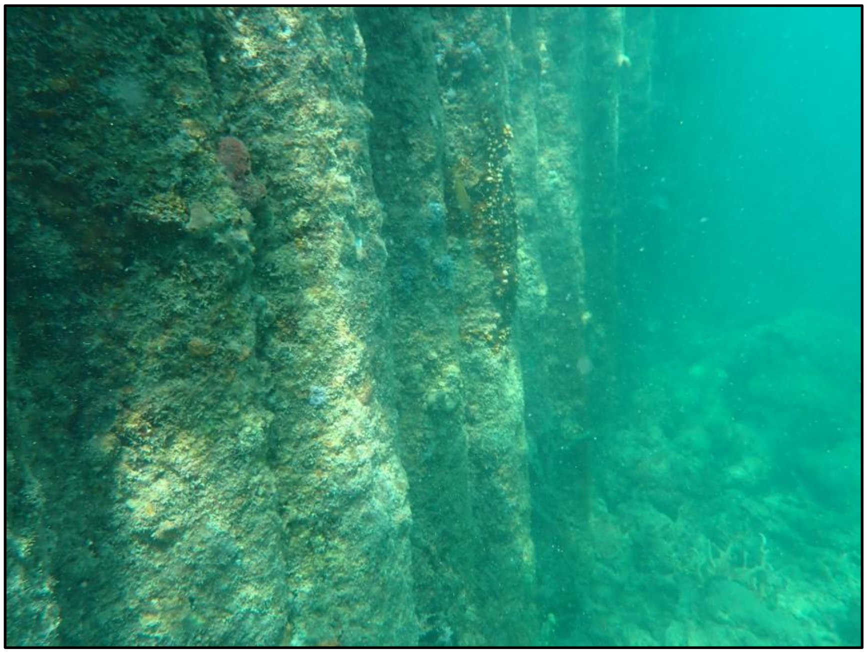


Photo 40: Typical view of steel sheet pile in Parcel 15

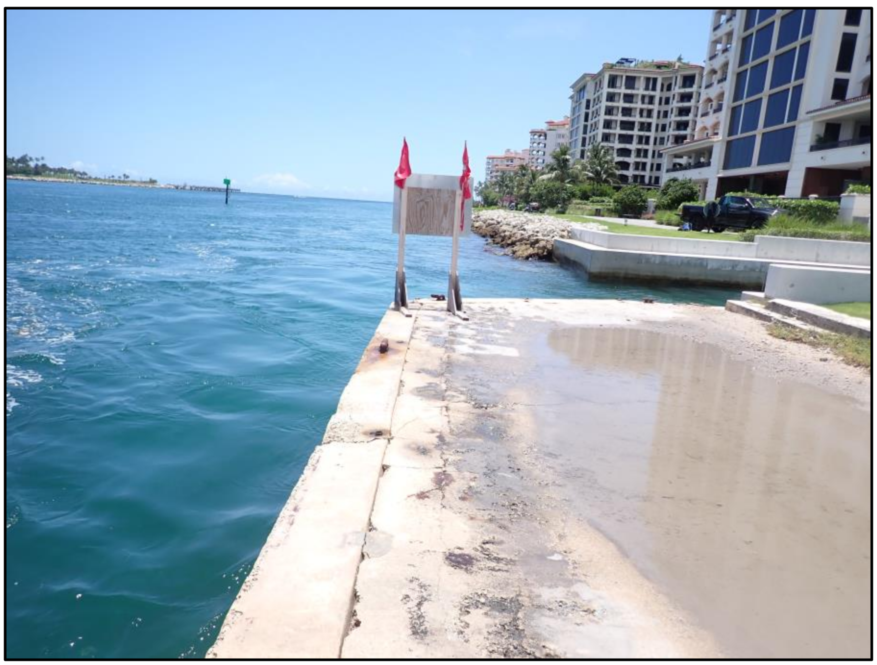


Photo 41: Topside view of Parcel 15

3.14 Section 13

Section 13 (formerly Task 2) is comprised of approximately 1644 linear ft of riprap revetment beginning at the end of the Section 12 basin and continuing east to the northeast corner of the island (**Figure 30**). From STA 0+00 to STA 8+90 the riprap is comprised of approximately 1-3 ft diameter limestone. At STA 8+90 the riprap changes to approximately 4-6 ft diameter angular granite stones and continues to the breakwater at the corner of the island (STA 16+44).

Figure 30: Section 13 Stationing Plan

Photo 42: Section 13 Smaller diameter limestone STA 0+00 to STA 8+90

Photo 43: Section 13 Large angular granite stones STA 8+90 to end of breakwater

Photo 44: General view of breakwater beyond eastern limit of Section 13

4 OBSERVATIONS

In general, the observed sections of shoreline were comprised of concrete panel/concrete pile wall, steel sheet pile, steel combi-wall, and riprap shoreline. The north, south and west shorelines, including Sections 3 through 13, are primarily protected from deep ocean waves, while the east shoreline, including Sections 1 and 2 are exposed. The north shoreline, including Sections 8 through 13, experiences heavy wake action due to vessel activity in Fisherman's Channel and Government Cut. The south shoreline, including Sections 3 through 8 experience light to moderate wake action due to vessels. Currents during the inspections were estimated to range from 0 knots to 4 knots. Underwater visibility varied throughout the inspections from 1-10 ft. Marine growth was observed on elements below the tidal zone with a hard layer up to 1-in thick. Corals were observed on shoreline elements of the majority of the sections and seagrasses were observed on bottom east of Section 1. Elements upland of each shoreline included landscaping, hardscaping, residential and marina structures including pools, as well as utilities.

The observations from the visual inspection are broken down by section below with representative photographs and relevant data included.

4.1 Section 1

The riprap within this section consists of stacked stable stones at an approximate 2:1 grade. Localized slopes up to 1:1 were observed at the western extent. No significant displacement, scour, or erosion was observed within the scope. The upland soil is well anchored with significant coverage of vegetation.

Photo 45: Localized 1:1 riprap slope at the western extent

4.2 Section 2

A review of the 2019 report prepared by Edgewater Resources, LLC titled, *Fisher Island Coastal Structure Assessment* revealed the length of Section 2 (South Breakwater) riprap was rated as “Good” with no deficiencies noted at the time.

The inspection performed by Cummins Cederberg found the Section 2 riprap displayed localized moderate voids in some locations due to unevenly stacked stones. Localized minor scour pockets were observed at the base of the riprap slope on the inshore face, assumed to be a byproduct of vessel traffic adjacent to the dock.

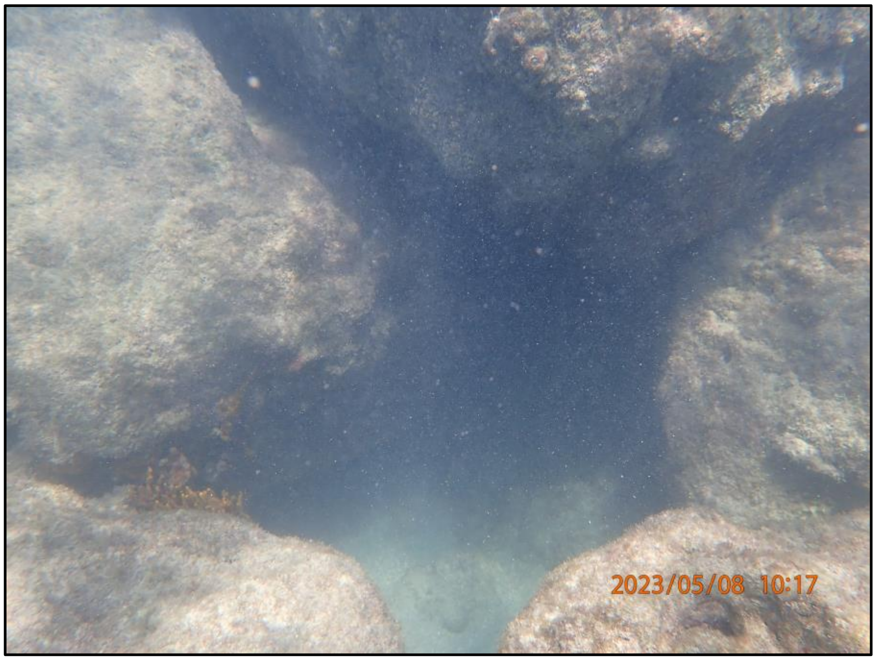


Photo 46: Section 2 localized moderate voids in the rip rap

Localized areas of uneven slope were observed where the stones had shifted from their original position leaving steeper slopes and loose stones at the base.

Photo 47: Section 2 localized riprap slope steeper than 1:1

Photo 48: Section 2 riprap shifted downward STA 0+55, facing northeast

At the beginning of the riprap adjacent to the dock the 6-in diameter steel outfall exhibits severe corrosion, see **Photo 49**.

Photo 49: Section 2 severe corrosion of steel outfall

4.3 Section 3

A review of the 2019 report revealed the Section 3 bulkhead (Guest Marina Bulkhead) included the notable observations listed below:

- Steel sheet pile between Station 53+05 and Station 66+25 is in Satisfactory condition overall with localized areas of minor to moderate deterioration, consisting of corrosion and pitting in the tidal zone of the sheet pile.
- Intermittent areas of subsidence and settlement of the pavers were observed along the backside of the bulkhead between Station 55+00 and 59+00, with more significant settlement up to 3” deep from Station 75+25 to 57+90.
- Below water, ultrasonic thickness measurements indicated only minor section loss in the steel.
- The concrete cap is in Satisfactory condition, with minor to moderate honeycombing and scaling along the top 2’ of the concrete cap throughout the length of the bulkhead.
- The concrete access platform at the south end of the bulkhead at Station 66+25 is in Satisfactory condition. The pile cap beneath the platform has spalled up to 60” long x 24” wide x 3” deep with exposed reinforcing steel.

The inspection performed by Cummins Cederberg found the Section 3 sheet piles appear to be significantly aged with the outer surface encased in corrosion byproduct and marine growth up to an inch thick. When cleaned the observed steel exhibits major to severe pitting. The UTM reading ranged from 0.223” to 0.343”. The original wall thickness is unknown but thickness readings with up to 35% variation indicate high levels of corrosion. Holes were observed at (3) locations throughout the scope. Observed steel pile deterioration appears much more severe relative to the 2019 Edgewater inspection.

Penetration into the upland fill measured up to 5 in. A significant tilt to the wall was observed at STA 0+00, which was not noted in the 2019 Edgewater report. An excavation was performed adjacent to this location to inspect the tie rod, which was found to be below the ground water level; therefore, only a tactile inspection was able to be performed. The tie rod appeared to be intact with moderate surface corrosion but no significant section loss.

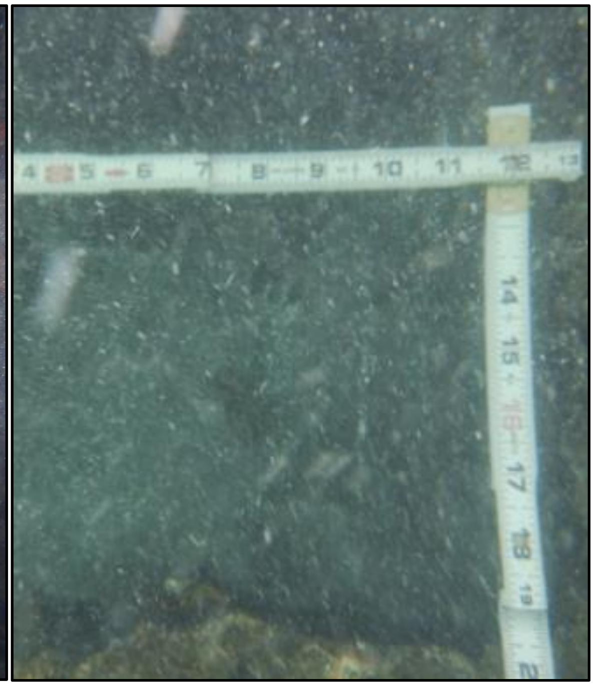
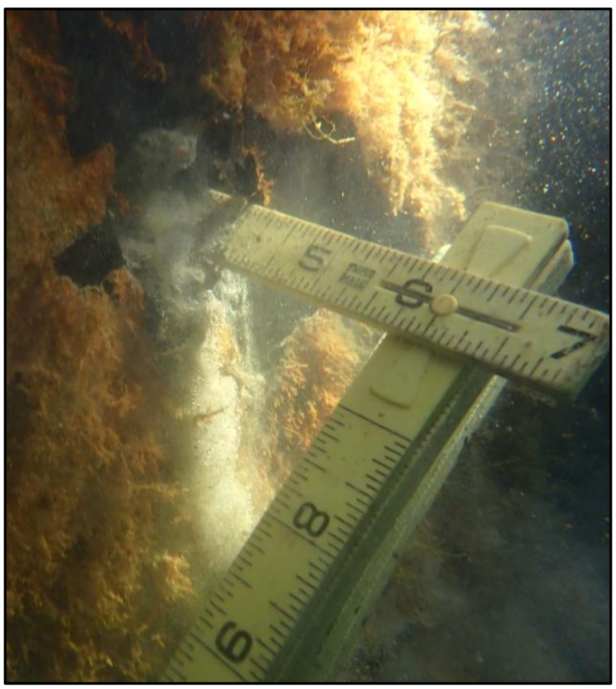


Photo 50: Typical view of cleaned steel sheet pile in Section 3, 2023 (left) and 2019 report (right)



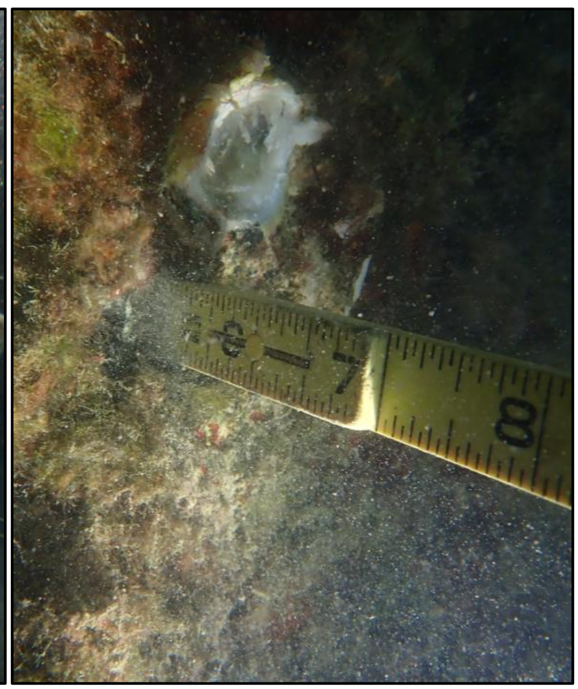


Photo 51: Multiple locations of 100% section loss in Section 3 steel sheet pile

From STA 0+00 to STA 3+30 the offshore edge of the concrete cap extends past the steel sheet pile. The bottom edge of the cap within these extents exhibits intermittent moderate to major spalling with exposed and corroded reinforcement with up to 100% section loss.

From STA 3+30 to STA 13+50 (Section 3 end) the offshore edge of the cap terminates at the inner flange of the steel sheet pile. In these locations the voids between the cap and the bulkhead are typically grouted but not for the full extent of the wall. In the locations where the voids were grouted, intermittent holes in the grout were observed.

The severe spall on the deck soffit near the east end of Section 3 as noted in the 2019 Edgewater report was observed and the reinforcement within the spall displayed up to full section loss indicating the deterioration has progressed. The above water portions of the cap exhibit localized areas of cracking up to 1/8-in. wide throughout. The paver settlement matched the description given in the 2019 report and the deterioration was not observed to have advanced significantly.

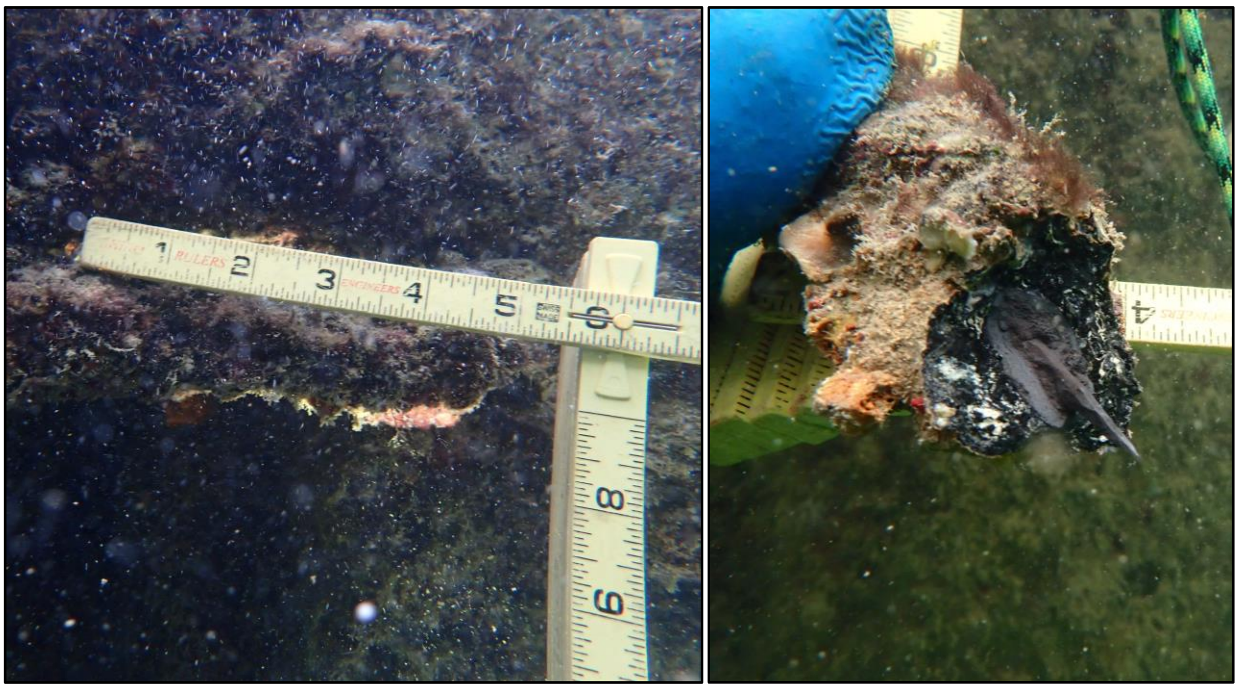


Photo 52: Major spalling of the bottom edge of the Section 3 concrete cap with full section loss of the reinforcement

Photo 53: Voids in between the offshore edge of the Section 3 cap and the bulkhead

Photo 54: Minor settlement of the pavers STA 4+70, 2023 (left) and 2019 report (right)

Photo 55: Severe spall in soffit slab adjacent to the Section 3 bulkhead termination

4.4 Section 4

A review of the 2019 report revealed the Section 4 bulkhead (South Bulkhead STA. 39+81 – STA. 49+89) included the notable observations listed below:

- The concrete sheet piles between Station 39+81 and Station 49+89 are in Fair condition. Minor subsidence was observed intermittently behind the wall with areas up to 10' long x 2' wide x 6" deep, attributed to migration of fines at the sheet pile joints. Gaps in the panel joints up to 1-1/2" wide were observed, with sediment accumulation on the channel bottom at the gaps. Approximately 50% of the panels have horizontal cracks in the splash zone, up to 1/8" wide by full length of the panels. Delaminated concrete was typically observed adjacent to the cracks, indicating corrosion-induced spalling.
- The concrete cap is in Satisfactory condition. The topside of the cap was recently repaired by routing and filling the cracks with cementitious repair compound. Hairline cracking has formed adjacent to the crack repairs in some locations, due to shrinkage of the repair product. However, no repairs are warranted at this time.

The inspection performed by Cummins Cederberg found the Section 4 concrete sheet piles exhibit widespread moderate to major spalling and cracking throughout. Most of the cracking appeared to be flexural in nature, occurring just below the cap. The sheet pile joints exhibit intermittent horizontal and lateral separation up to 2 in. Intermittent sinkholes were observed upland, in line with the sheet pile joints. Similarly, intermittent sediment accumulation was observed at the seabed adjacent to the joints. From STA 5+70 to STA 6+35, the wall deflected visibly outwards. A tie rod was inspected adjacent to this area, at approximately STA 6+65. The rod was found to exhibit moderate corrosion of the outer layer, with a uniform circumferential surface of the solid remaining steel. In general, the concrete sheet pile deterioration has progressed since the 2019 Edgewater inspection.

Photo 56: Widespread spalling and cracking of the Section 4 concrete sheet pile

Photo 57: Section 4 STA. 5+72 Typical view of lateral separation (left) and horizontal separation (right) at sheet pile joints

Photo 58: Section 4 Typical view of upland sinkhole in line with sediment accumulation at mudline

At two locations the bottom face of the concrete cap has spalled, exposing severely corroded reinforcement. The top of the cap exhibits localized cracking, less than 1/8-in. wide, with widespread crack repairs throughout.

Photo 59: Severe spalling of Section 4 concrete cap with severe corrosion of the exposed reinforcement around STA 8+90.

Each of the 14-in. diameter corrugated steel outfalls exhibits partial blockage of the outfall from sediment accumulation. Severe corrosion of the corrugation was also observed.

Photo 60: Section 4 STA. 0+30 Typical partial blockage of corrugated steel outfall with severe corrosion in the tidal zone

Severe upland settlement was observed at the western bulkhead termination. A cavity was noted at the corner junction with Section 5, facilitating backfill loss.

4.5 Section 5

A review of the 2019 report revealed the Section 5 bulkhead (Private Marina Bulkhead STA. M0+00 – STA. M3+00, STA. M21+60 – STA. M22+60) included the notable observations listed below:

- The concrete soldier piles are in Poor condition. Approximately 90% of the soldier piles have corrosion induced cracking or delamination in the tidal and splash zone. Below water, no significant deterioration was observed on the soldier piles; however, small sediment deposits were typically observed at the joints between the soldier piles and shutter panels, indicating loss of fill through the joints.
- The concrete shutter panels are in Satisfactory condition. Moderate subsidence was observed intermittently behind the wall, with isolated locations of sinkholes

and undermining of adjacent site features, attributed to migration of fines through the panel joints behind the soldier piles.

- The concrete cap is in Satisfactory condition. Hairline to 1/16” wide cracks typically spaced at 5’ to 7’ O.C. and minor intermittent mechanical spalls were located along the top of the cap.
- The private marina entrance jetties are in Satisfactory condition. No significant deterioration was observed

The Section 5 inspection performed by Cummins Cederberg found the southern 20 ft and the northern 10 ft on the east and west bulkheads present approximately 6-10 ft of exposed wall below the cap. The interior lengths of bulkhead present approximately 1-2 ft of exposed wall below the cap. The exposed lengths of the concrete king piles typically exhibit severe deterioration in the form of widespread 1/8-in. to 1-in. wide cracks with heavy rust staining and localized delamination of the pile face. The condition of the piles below the line of riprap is unknown. The king pile observations align with those noted in the 2019 Edgewater report with the addition of observed king pile deterioration below the water line.

The concrete pile cap exhibits intermittent moderate to major deterioration in the form of 1/8 to ½ in. cracking and localized spalling. Concrete cap deterioration appears to have progressed since the 2019 Edgewater report.

Photo 61: Section 5 Typical severe deterioration of concrete king piles with typical major deterioration of the concrete pile cap

At STA 0+00 on the east wall a 1 ft deep sinkhole was present at the Section 4 termination. A tie rod attached to the Section 4 wall was exposed and exhibits severe corrosion and significant loss of section. At the mudline large scale voids are present between the Section 4 termination pile and the first king pile at Section 5. It is unclear what prevents the remaining upland soil from evacuating through the voids at the base, but fill loss is assumed to continue.

Photo 62: 1 ft deep sinkhole at Section 5 STA. 0+00 E (left) with exposed tie rod attached to Section 4 bulkhead (right)

Photo 63: Large scale voiding adjacent to mudline at interface between Section 4 and Section 5 bulkheads

The Section 5 bulkhead at this location was constructed concurrently with the bulkhead within the Resident Marina in 1978. Due to a recent bulkhead collapse during construction efforts within the Resident Marina, which was due to tie rods failure, the tie rods within this section were a high priority to inspect. Three locations were chosen including at STA. 0+05 E, STA. 2+75 E, and STA. 1+00 W. At STA 0+05 E the tie rod was found to be significantly bent with moderate corrosion of the outer layers. At STA 2+75 E the tie rod was fully broken with evidence of severe corrosion and no longer provides lateral support to the bulkhead. At STA 1+00 W the tie rod was in-tact and in plane with moderate corrosion of the outer layers.

The riprap lining the Section 5 bulkheads and comprising the jetties was found to have a stable slope throughout. Initial mangrove vegetation has taken hold between the riprap along the bulkhead. The riprap maintains contact with the bulkhead providing much needed lateral stabilization for the wall.

Photo 64: Section 5 Failed tie rod at STA. 2+75 E (left) and bent tie rod at STA. 0+05 E (right)

4.6 Section 6

A review of the 2019 report revealed the Section 6 bulkhead (South Bulkhead STA. 22+00 – STA. 37+34) included the notable observations listed below:

- The concrete solder piles between Station 22+00 and 37+34 are in Fair condition. Concrete pile encasements were previously installed on the soldier piles in the tidal and splash zone, which have partially failed on approximately 80% of the piles. Below water, no significant deterioration was observed on the soldier piles.
- The concrete shutter panels between Station 22+00 and 37+34 are in Fair condition. Minor subsidence was observed intermittently behind the wall, attributed to migration of fines through the panel joints behind the soldier piles. Approximately 15% of the panels have horizontal cracks in the splash zone up to 1/8" wide by the full length of the panels, with delaminated concrete typically adjacent to the cracks indicating corrosion-induced spalling.
- The concrete cap is in Satisfactory condition. The topside of the cap was recently repaired by routing and filling the cracks with cementitious repair compound Hairline cracking has formed adjacent to the crack repairs in some locations due to shrinkage of the repair product; however, no repairs are warranted at this time.

The Section 6 inspection performed by Cummins Cederberg found the concrete pile encasements exhibit widespread failures within the tidal zone and below and localized full height delamination of the offshore face. From previous involvement with Fisher Island projects, we understand this to be a construction defect caused by washout of the concrete while it set. The exposed pile within the tidal zone and below was heavily obscured by residual grout and marine growth and no significant deficiencies were observed. The pile conditions were in line with those observed in the 2019 report. The concrete panels exhibit widespread 1/8 in. wide horizontal or diagonal cracking. Significantly more panels displayed observed deterioration than listed in the 2019 report. Widespread crack repairs were present throughout the concrete cap in line with the description of the cap in the 2019 report. One tie rod was excavated within Section 6 at approximately STA 8+40. The tie rod was below the existing water line at the time of inspection so a visual inspection could not be performed, and the rod was inspected tactilely instead. The tie rod felt smooth with no significant corrosion.

Photo 65: Typical washout of concrete pile encasement (left) and localized delamination of offshore extent of pile encasement

Photo 66: Typical 1/8 in. cracking of the concrete panels

Below the water line intermittent gaps were found between the pile and the panel 2-4 in. wide. This exposed the pile gap and lateral separation of the panels up to 2 in. wide with 4-12 in. of penetration into the joint were measured. Intermittent signs of freshwater intrusion were present at the pile panel interface. Intermittent upland sinkholes were observed in line with the panel joints.

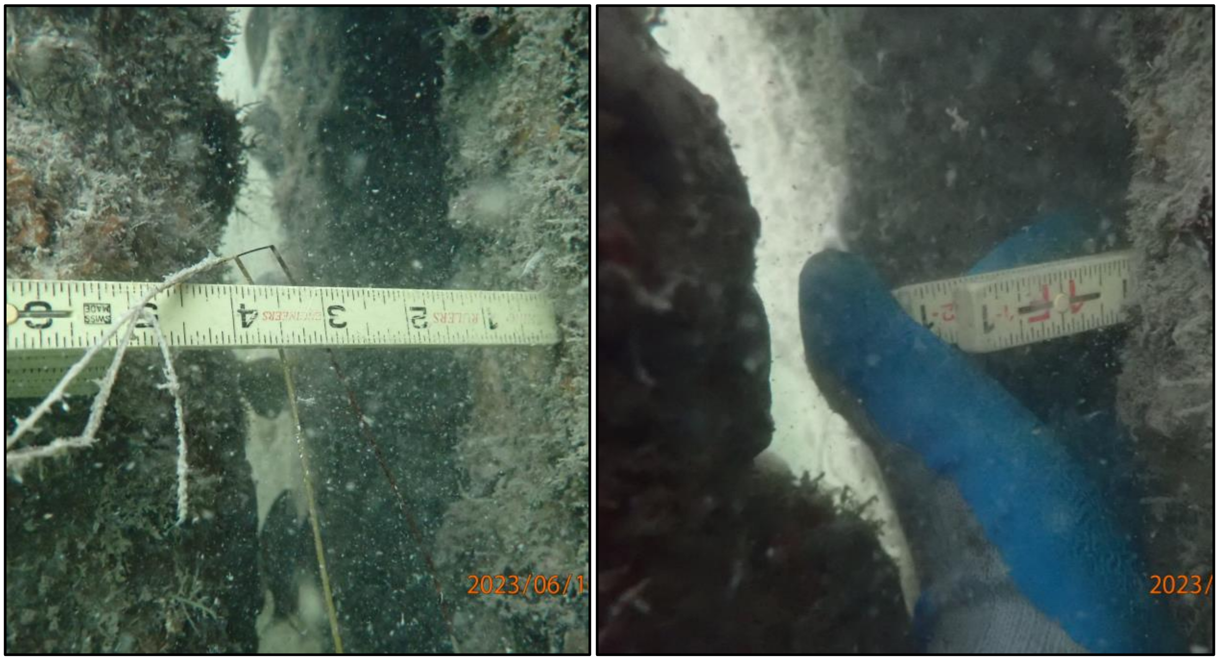


Photo 67: Typical pile panel gap with significant penetration into the panel joint

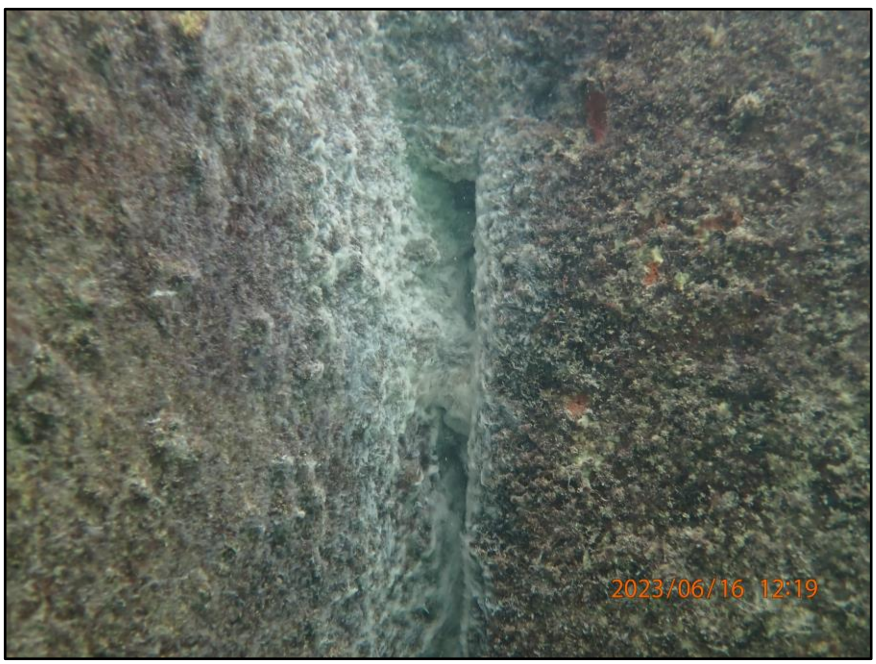


Photo 68: Typical evidence of freshwater intrusion at the pile panel interface

Photo 69: Typical upland sinkhole

4.7 Section 7

A review of the 2019 report revealed the Section 7 bulkhead (Northwest Bulkhead STA. 21+05 – STA. 22+00) included the notable observations listed below:

- The steel sheet pile from Station 21+05 to Station 22+00 is in Satisfactory condition with moderate deterioration observed, consisting of corrosion, scaling, and minor section loss in the tidal and splash zone. Below water, the coating is fully intact and ultrasonic thickness measurements indicated no significant section loss.
- The concrete cap between Station 21+05 and Station 22+00 is in Satisfactory condition, with random hairline cracking in the underside of the crack observed. The topside of the cap was recently repaired by routing and filling the cracks with cementitious repair compound. Hairline cracking has formed adjacent to the crack repairs in some locations due to shrinkage of the repair product; however, no repairs are warranted at this time.

The Section 7 inspection performed by Cummins Cederberg found the steel sheet pile exhibits severe corrosion throughout the tidal zone and splash zone with ½ – 1 in. of heavy blistering. When cleaned the exposed steel exhibits major to severe pitting. UTM readings were taken at approximately STA 0+55. The average thickness ranged from 0.285 in. thick at the outer flange tidal zone reading and 0.383 at the mudline outer flange. The flanges were typically thinner in the tidal zone than the mudline indicating typical accelerated corrosion rates in the tidal zone. Since the original section and date of construction is unknown the rate of corrosion is also unknown. If UTM readings are taken in a future inspection it can be compared to the current readings to estimate corrosion rates. The underside of the cap exhibits intermittent 1/8 in. cracking adjacent to the steel sheet pile, likely due to the expansion of the sheets caused by severe corrosion. The topside of the cap exhibits intermittent grouted crack repairs. The cap appears to be of a more recent construction than the existing sheet pile.

Photo 70: Typical condition of steel sheet pile and concrete cap

Photo 71: STA 12+18 Cleaned section of steel sheet pile for UTM reading with major to severe pitting

At STA 0+00 the Section 8 transition exhibits 6-12 in. of sediment accumulation at the mudline. At approximately STA 0+50 a sinkhole is present at the bend in the bulkhead measured 32 in. long by 16 in. wide by 21 in. deep. One tie rod was investigated at approximately STA 0+70. The tie rod was encased in a painted outer coating that appeared to be in like new condition. It is assumed that the tie rods have been replaced since the original construction.

Photo 72: Sediment accumulation at Section 8 termination

Photo 73: STA 12+18 Sinkhole and in line joint at corner of bulkhead

Photo 74: Tie rod inspected at STA 0+70 in like new condition

4.8 Section 8

A review of the 2019 report revealed the Section 8 bulkhead (Northwest Bulkhead STA. 9+40 – STA. 21+05) included the notable observations listed below:

- The steel sheet pile from Station 9+40 to 21+05 is in Good condition with minor deterioration observed, consisting of 10-15% coating loss in the splash zone with light surface corrosion of the exposed steel. No significant section loss was observed. Below water, the coating was fully intact and ultrasonic thickness measurements indicated no significant section loss.
- The prestressed concrete batter piles (Section 9+00 to Station 12+45 and Station 16+00 to Station 21+05) are in Good condition. No significant deterioration was observed.
- The concrete cap between Station 9+40 and 21+05 is in Good condition with minor deterioration observed. The cap has hairline transverse cracks typically spaced at 5’ to 7’ O.C, and one mechanical spall at Station 18+00, approximately 4’-6” long x 1’ wide x 6” deep. The hairline cracks are typical shrinkage cracks associated with concrete construction and do not warrant repairs at this time.

All steel sheet pile sections within Section 8 were installed during the 2012 repair effort. The Section 8 inspection performed by Cummins Cederberg found the sheet pile within the tidal and splash zones exhibits moderate rust staining and blistering with intact coating. Anodes were found installed at the top of the sheet pile located at either the inner flange or outer flange intermittently. Jet filters were installed within the tidal zone on the inner flange throughout. Intermittent jet filters exhibit moderate to major corrosion. No significant deterioration was observed on the concrete battered piles.

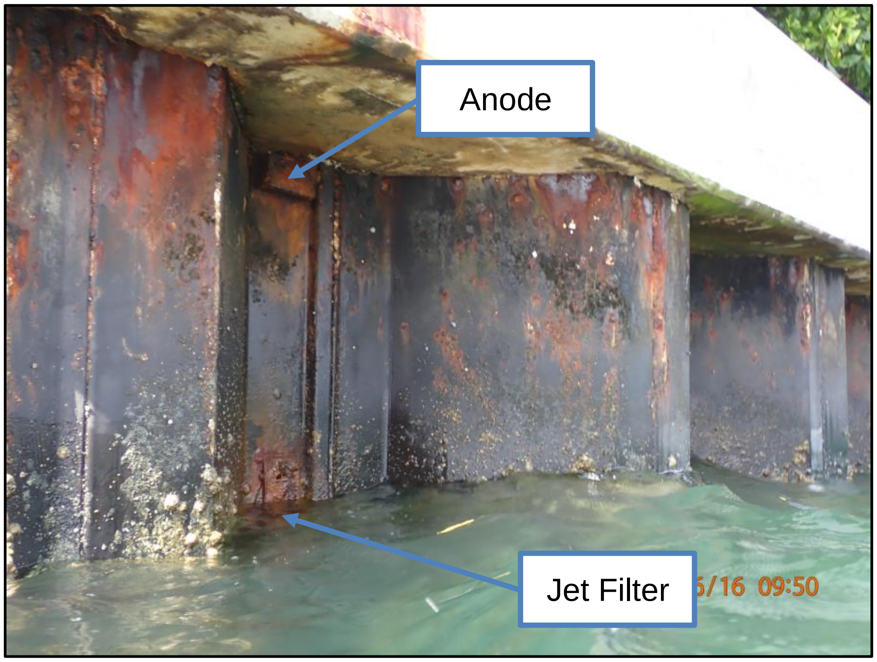


Photo 75: Moderate corrosion of the steel sheet pile within the tidal zone with localized major corrosion of the jet filter

Localized hairline to 1/8 in. cracking with intermittent rust staining was observed on the underside of the concrete cap throughout. From STA 6+64 to Section 7 the concrete cap exhibits widespread 1/8 in. cracking with localized major spalling at the underside of the cap. Localized transverse cracking was observed on the top of the cap with intermittent epoxy injection crack repairs.

Photo 76: Localized major spalling with widespread cracking of the concrete cap

Intermittent sinkholes were observed directly inshore of the cap. These sinkholes were documented and can be found in Table 1.

Table 1: Sinkholes

Station	Length	Width	Depth
1+30	6 in.	6 in.	14 in.
1+40	9 in.	9 in.	4 in.
7+60	12 in.	4 in.	6 in.
8+00	14 in.	6 in.	9 in.
12+18	32 in.	16 in.	21 in.
17+75	20 in.	34 in.	20 in.
22+50	22 in.	34 in.	24 in.

Photo 77: Typical view of Section 8 sinkhole

4.9 Section 9

A review of the 2019 report revealed the Section 9 bulkhead (North Bulkhead STA. 0+00 – STA. 9+40) included the notable observations listed below:

- The steel sheet pile has 100% coating failure with moderate corrosion and section loss in the tidal and splash zone. Below water, the coating was approximately 50% intact; minor surface corrosion and pitting was observed on exposed steel with ultrasonic thickness measurements indicating less than 15% steel section loss.
- Hairline cracks are located throughout the top of the cap, typically at 3’ to 5’ spacing, and intermittent mechanical spalls are located along the outer edge of the concrete cap, likely due to barge impacts. One larger 10’ long x 14” wide x 5” deep mechanical spall with exposed rebar was also observed, located at Station 2+30.

The Section 9 inspection performed by Cummins Cederberg found the steel sheet pile from STA 0+00 – STA 0+30 typically exhibits severe corrosion in the splash and tidal zones with heavy buildup of corrosion byproduct. The cap was poured continuously with the Section 10 bulkhead cap and is assumed to have been constructed during the 2021 basin combi-wall construction. At STA 0+00 at the mudline a void is present measured

18 in. high and 4 in. wide with loose fill accumulating at the base of the void. No sinkhole was observed topside, and the grade was likely evenly graded during the 2021 construction effort. At the sheet pile connection at STA 0+30 the vertical joint exhibits signs of freshwater intrusion throughout.

Photo 78: Typical condition of steel sheet pile (left) and concrete cap (right) STA 0+00 – STA 0+30

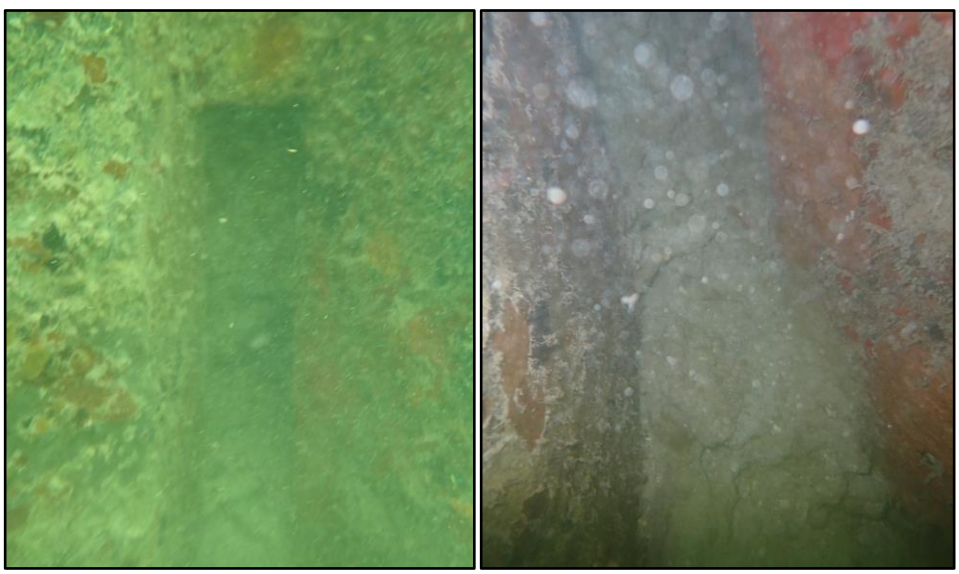


Photo 79: Voiding at mudline at STA 0+00 (Section 10 interface)

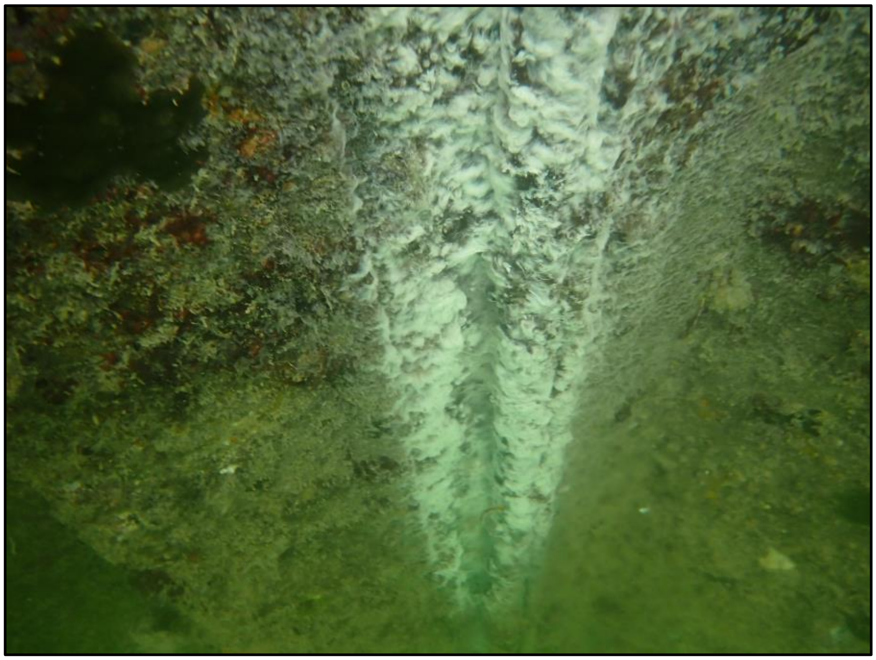


Photo 80: Freshwater intrusion at STA 0+30 sheet pile interface

The sheet pile between STA 0+30 and STA 0+85 exhibits minor corrosion and rust staining bleeding through the coating within the top 3 ft. The concrete cap exhibits intermittent moderate cracks with rust staining less than 1/8 in. wide, intermittent minor to moderate erosion spalls, and minor discoloration throughout.

Photo 81: Typical condition of steel sheet pile and concrete cap STA 0+30 – STA 0+85

The sheet pile between STA 0+85 and STA 4+64 exhibits moderate corrosion within the top 3 ft. Minor degradation of the anodes was observed. Moderate corrosion of the steel plates, hex bolts, and tie rods was observed. From STA 0+85 to STA 3+00 intermittent moderate spalling up to 6 in. deep and 5 ft long was observed on the offshore edge of the top of cap. Intermittent cracking up to 1/8 in. wide was observed throughout.

Photo 82: Typical condition of steel sheet pile and concrete cap STA 0+85 – STA 4+64

The sheet pile between STA 4+64 and STA 9+36 exhibits minor rust staining within the top 3 ft of the sheet pile. Localized moderate to major corrosion of the jet filters was observed. Minor degradation of the anodes was observed. One major spall of the pile cap was observed at the STA 4+64 interface. Intermittent 1/8 in. cracking was observed throughout the pile cap.

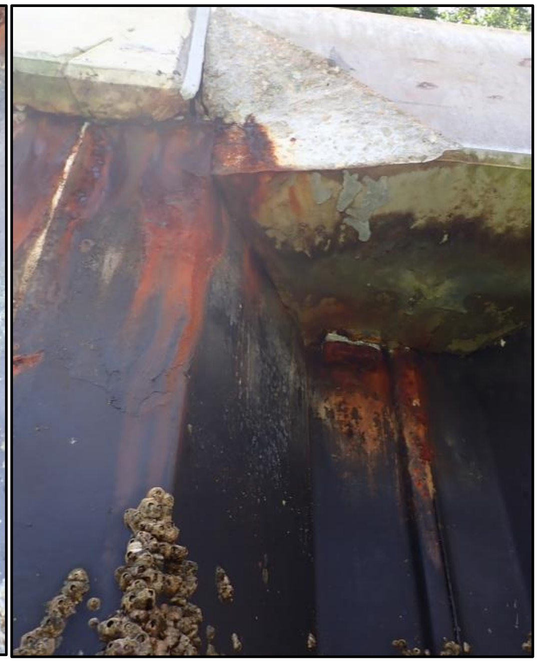


Photo 83: Typical condition of steel sheet pile STA 4+64 – STA 9+39 and major cap spall at STA 4+64

Photo 84: Localized major corrosion of jet filter

4.10 Section 10

The steel combi-wall was installed in 2021 and therefore comparison to the observed conditions in the 2019 report cannot be made. The Section 10 inspection performed by

Cummins Cederberg found the combi-wall exhibits minor corrosion of the HZ1080M piles and minor rust staining bleeding through the coating of the sheet pile. Intermittent minor transverse hairline shrinkage cracks were observed throughout the concrete cap.

Photo 85: Minor corrosion of HZ piles and minor rust staining of AZ sheet pile

Photo 86: Localized shrinkage cracks

At STA 4+70 where the newly installed combi-wall intersects with the aging steel sheet pile a 1 ft high 3 in. void is present at the mudline. The sheet pile from STA 4+70 to STA 5+15 exhibits severe corrosion throughout. The concrete cap has failed from STA 4+70 to approximately STA 4+90 and exhibits severe spalling and general deterioration for the remainder.

Photo 87: Void at bulkhead connection at mudline STA 4+70

Photo 88: Failed cap from STA 4+70 to 4+90, severe corrosion of the steel sheet pile

4.11 Section 11

The newer bulkhead in Section 11 was installed landward of the original. As a result, only minimal areas of the new bulkhead sections were able to be observed. The steel sheet pile was installed in 2021 and, therefore, comparison to the observed conditions in the 2019 report cannot be made. The Section 11 inspection performed by Cummins Cederberg found the concrete cap from STA 0+00 to STA 0+84 exhibits minor to moderate delamination of the finish within the tidal zone, as well as minor to moderate discoloration and erosion throughout. The concrete cap from STA 0+84 to STA 5+80 exhibits widespread major to severe spalling of the concrete cap throughout, particularly to the bull rail. The newly installed steel sheet pile was not visible throughout the majority of the inspection. At STA 3+60, a 3-ft high by 9-ft wide void was observed in the sheets just below the cap, revealing newer sheet pile driven approximately 2-3 ft landward. Access was limited, but one UTM reading was taken on the outer flange of the newer sheet pile, measured at 0.295 in. No significant deficiencies were observed on the sheet pile from STA 5+80 to STA 9+60. When the marine growth was cleared, the coating remained clean and intact. Localized moderate spalling was observed on the bottom edge of the concrete cap.

Photo 89: Intermittent delamination of the concrete cap STA 0+00 to STA 0+84

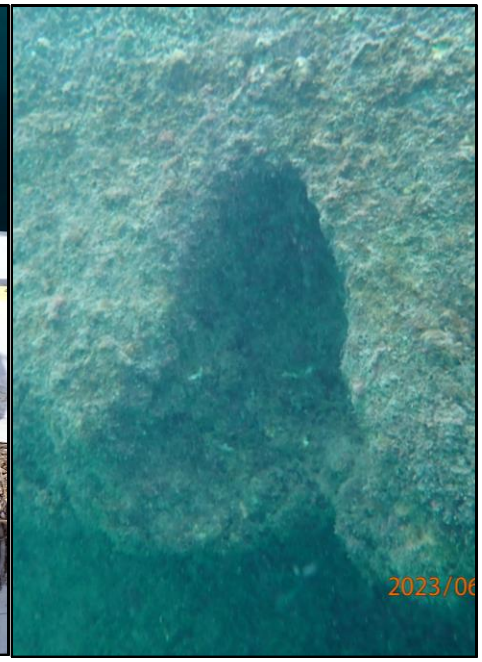
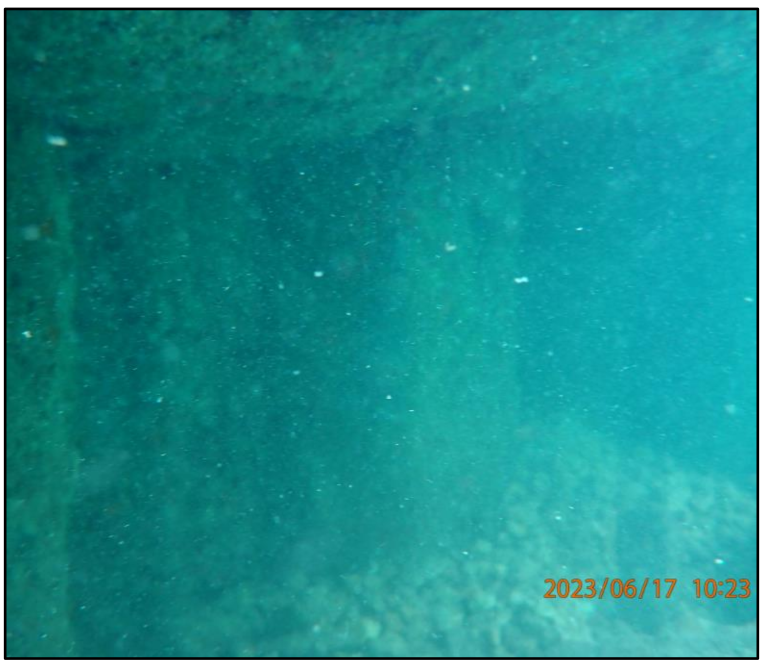


Photo 90: Widespread major to severe spalling of the concrete cap, STA 0+84 to STA 5+80



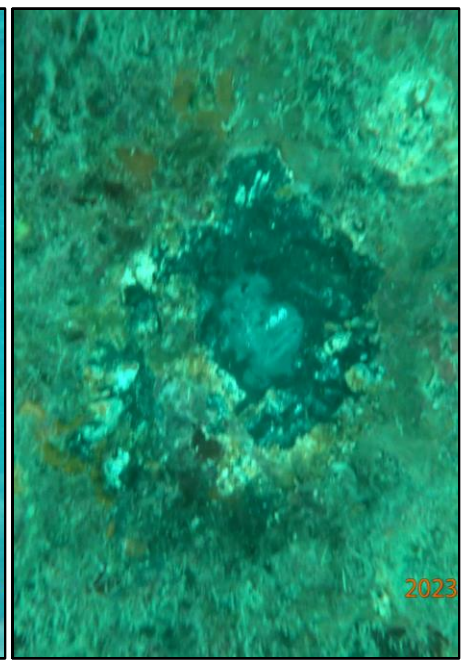


Photo 91: View of inner layer of sheet pile visible through large void in outer layer of sheet pile at STA 3+60 (left); cleaned area of sheet pile for UTM (right)

Photo 92: Typical condition of steel sheet pile and concrete cap (left) and localized moderate spall of concrete cap (right), STA 5+80 to STA 9+60

4.12 Section 12

The newer bulkhead in Section 12 were installed landward of the original bulkhead; therefore, only minimal extents of the new bulkhead sections were able to be observed. During the 2019 inspection no portions of the seawall were observed to be visible. The Section 12 inspection performed by Cummins Cederberg found the concrete cap exhibits intermittent transverse 1/8 in. to hairline cracks throughout. One location of cracking ¼ in. wide was observed at STA 0+00 forming a closed spall. Localized areas of minor abrasion spalling were observed on the concrete edges. The steel combi wall driven landward of the original wall was intermittently visible through voids in the sheet pile. At all observed Locations the coating was observed to be clean and intact. At STA 2+90 a sinkhole was observed upland of the construction joint measured 1 ft wide and 1 ft deep. Signs of freshwater intrusion were observed throughout the full height of the construction joint.

Photo 93: Intermittent cracking of the concrete cap up to 1/8 in. with localized cracking up to ¼ in. forming a closed spall

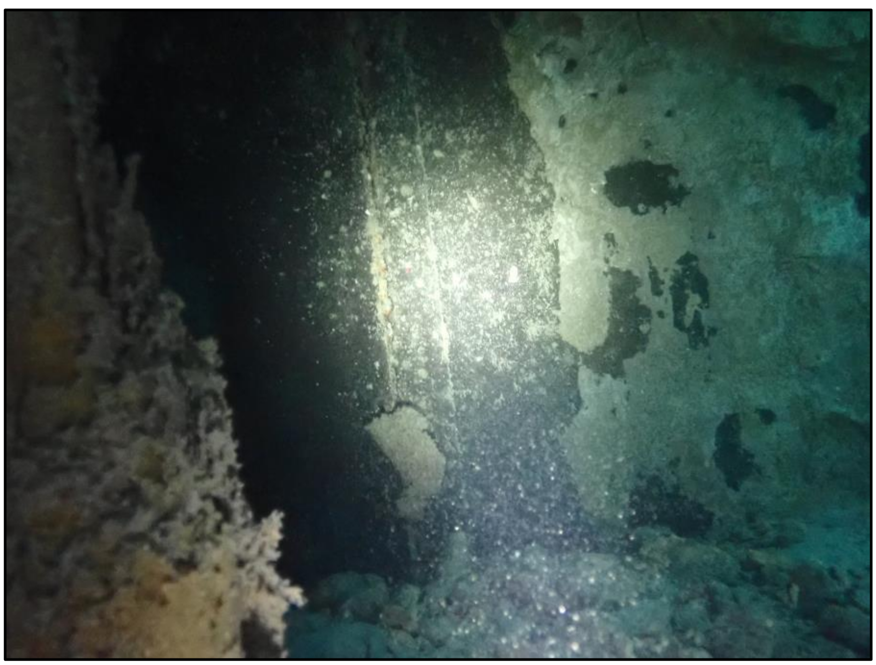


Photo 94: Typical steel surface of combi-wall with clean and intact coating

Photo 95: Sinkhole at northeastern termination (STA 2+90) with signs of freshwater intrusion visible at the in-water portion of the joint

4.13 Parcel 15

A review of the 2019 report revelated the Parcel 15 sheet pile included the notable observations listed below:

- The steel sheet pile is in **Poor** condition with moderate to advanced deterioration throughout, consisting of heavy corrosion in the tidal and splash zone of the sheeting with up to 30% section loss and isolated corrosion holes.
- The concrete cap is in **Poor** condition with widespread cracking and spalling in the concrete cap, exhibiting exposed and corroded reinforcing steel. Portions of the cap have significant section loss due to large spalls in localized areas.

These observations were in reference to the bulkhead from the termination of Section 12 to the northeast corner of the Section 11 Ferry basin. The seawall at the termination of the specified scope (STA 0+84) has been replaced since the previous inspection.

The Parcel 15 inspection performed by Cummins Cederberg found the steel sheet pile to display widespread major to severe buildup of corrosion byproduct that when cleaned displayed severe pitting of the steel. Freshwater intrusion and signs of fill loss were observed at the Section 12 termination (STA 0+00) and a detailed description and photos can be found in the Section 12 observations. Voiding at the mudline of the STA 0+84 termination into new sheet pile was observed approximately 12 in. high and 1 in. wide with 1.5 ft of measured penetration into the joint. Widespread major to severe spalling of the below water portions of the cap and intermittent major to severe spalling of the above water portions of the cap adjacent to severely corroded cleat fixtures was observed. Severe spalling of the cap of the western wall was observed throughout. Widespread moderate erosion and spalling was present on the remainder of the cap. The upland platform formed a significant depression with pooling observed throughout.

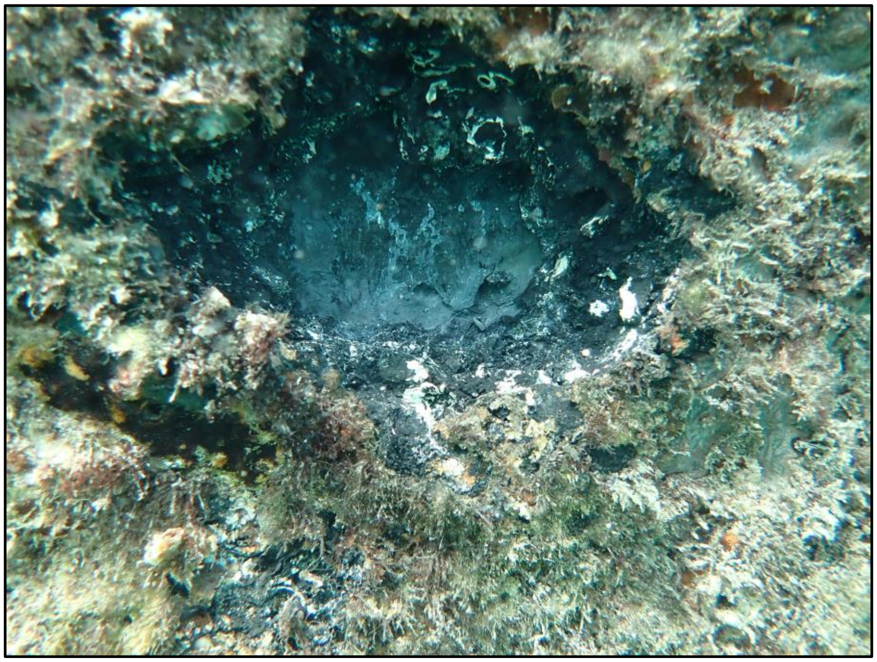


Photo 96: Severe pitting of the Parcel 15 steel sheet pile observed when cleaned

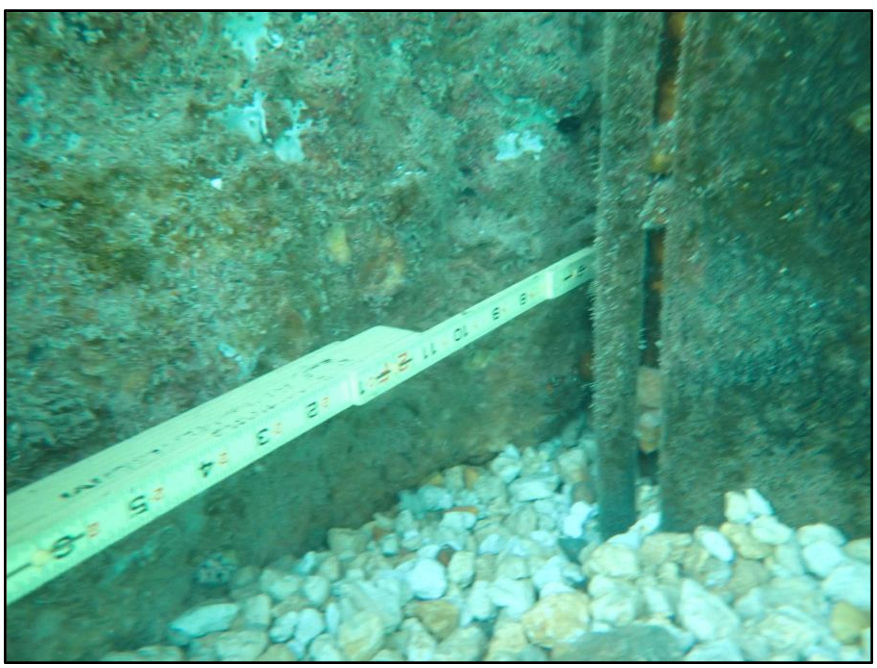


Photo 97: 1.5 ft of penetration into voiding at STA 0+84 sheet pile termination

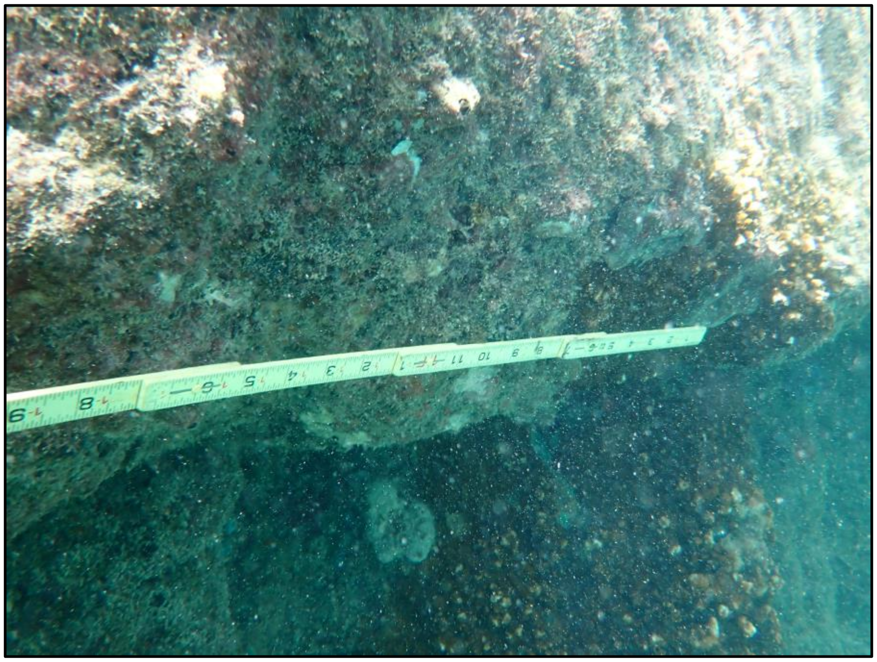


Photo 98: Widespread severe spalling of the bottom edge of the concrete cap


Photo 99: Intermittent severe cracking and spalling on offshore face due to severely corroded embedded cleats

Photo 100: Typical widespread moderate erosion and spalling of the offshore face of the cap on the north face (left) and severe spalling of the cap on the west face (right)

Photo 101: Widespread upland pooling throughout Parcel 15

4.14 Section 13

A review of the 2019 report revealed the Section 13 riprap (North Revetment and Jetty) included the notable observations listed below:

- The riprap revetment appears stable and is in Satisfactory condition with minor deterioration observed, consisting of erosion to the shoreline immediately behind the revetment between Station NR9+00 and Station NR16+00. The erosion is primarily due to wave action penetrating through the gaps in the large stone and gradual degrading the shoreline embankment.

The Section 13 inspection performed by Cummins Cederberg found the riprap from STA 0+00 to STA 8+90 exhibits intermittent locations of grades steeper than 1:1 above the waterline in line with grades shallower than 3:1 below the water line indicating moderate washout and movement of the stones from their original position. Localized areas of minor erosion of the soil upland of the stones were also observed. Mangroves and other large vegetation were present from STA 8+90 to the end anchoring the soil and preventing erosion. No shifting of the large diameter stones within this section was observed.

Photo 102: Intermittent grades of smaller diameter riprap at steeper than 1:1 slope

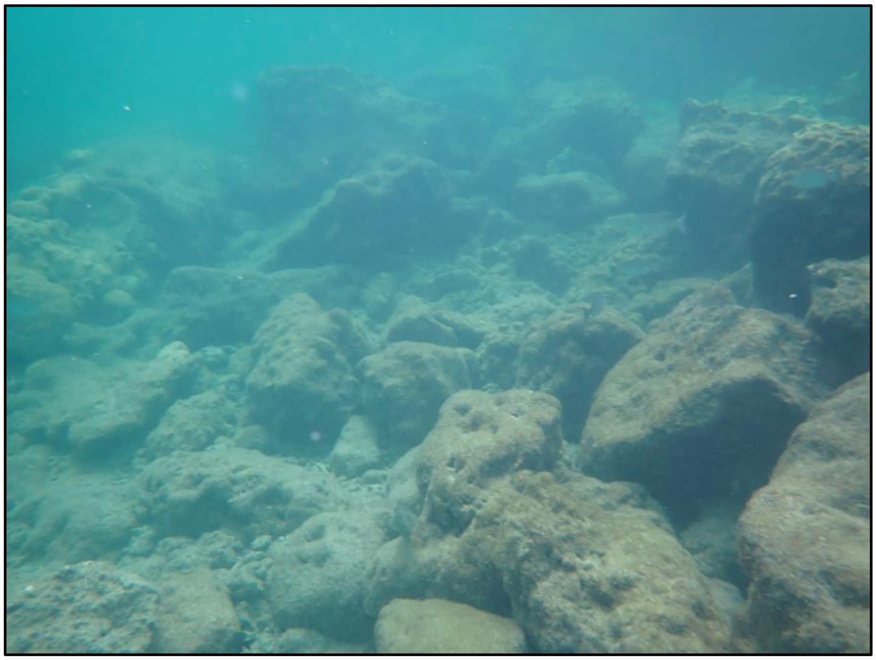


Photo 103: Below water riprap in line with above water steeper grades indicating movement of the stones from their original position


Photo 104: Localized areas of minor erosion of the upland fill

4.15 Ultrasonic Thickness Measurements

Along the sections comprised of steel sheet piles, (10) locations of Ultrasonic Thickness Measurements UTM readings were taken. Three readings were taken on the outer flange, web, and inner flange of the sheet pile at the mudline, mid-water, and waterline. Corrosion rates typically vary between the different zones as shown in **Figure 2,** so this gives a good representative sample of the submerged portions of the wall. Section 10 is comprised of a combi-wall section and additional readings were taken at the steel HZ piles. The flange of the HZ piles is tapered so the readings varied depending on the location within the flange that was measured. The reading at Section 11 was taken on the old waterward sheet pile, as access to the new sheets behind was not available in the basin. Outside the basin, the channel current at the time of the inspection was not conducive for readings on the newer sheets. UTM readings are summarized in **Table 2** below.

Table 2: Ultrasonic Thickness Measurements

Section	STA	Elevation	Location	1	2	3	AVG
3	6+80	Waterline	Outer Flange	0.220	0.225	0.225	0.223
			Web	0.220	0.225	0.225	0.223
			Inner Flange	0.290	0.295	0.295	0.293
		Mid-Water	Outer Flange	0.340	0.345	0.345	0.343
			Web	0.270	0.275	0.275	0.273
			Inner Flange	0.320	0.320	0.320	0.320
		Mudline	Outer Flange	0.235	0.235	0.230	0.233
			Web	0.330	0.325	0.335	0.330
			Inner Flange	0.290	0.295	0.295	0.293
7	0+55	Waterline	Outer Flange	0.285	0.285	0.285	0.285
			Web	0.330	0.330	0.330	0.330
			Inner Flange	0.300	0.300	0.305	0.302
		Mid-Water	Outer Flange	0.365	0.365	0.355	0.362
			Web	0.360	0.360	0.365	0.362
			Inner Flange	0.355	0.355	0.355	0.355
		Mudline	Outer Flange	0.350	0.350	0.450	0.383
			Web	0.330	0.330	0.320	0.327
			Inner Flange	0.350	0.350	0.350	0.350
8	0+25	Waterline	Outer Flange	0.550	0.565	0.570	0.562
			Web	0.410	0.410	0.410	0.410
			Inner Flange	0.565	0.565	0.560	0.563
		Mid-Water	Outer Flange	0.560	0.560	0.560	0.560
			Web	0.415	0.415	0.415	0.415
			Inner Flange	0.565	0.565	0.570	0.567
		Mudline	Outer Flange	0.555	0.560	0.550	0.555
			Web	0.410	0.410	0.410	0.410
			Inner Flange	0.565	0.565	0.570	0.567
8	7+00	Waterline	Outer Flange	0.370	0.370	0.365	0.368

			Web	0.360	0.360	0.355	0.358
			Inner Flange	0.360	0.365	0.360	0.362
		Mid-Water	Outer Flange	0.350	0.350	0.350	0.350
			Web	0.350	0.300	0.355	0.335
			Inner Flange	0.360	0.360	0.360	0.360
		Mudline	Outer Flange	0.340	0.340	0.345	0.342
			Web	0.360	0.360	0.360	0.360
			Inner Flange	0.360	0.360	0.355	0.358
9	3+50	Waterline	Outer Flange	0.655	0.665	0.650	0.657
			Web	0.460	0.460	0.460	0.460
			Inner Flange	0.645	0.650	0.650	0.648
		Mid-Water	Outer Flange	0.655	0.655	0.655	0.655
			Web	0.450	0.455	0.455	0.453
			Inner Flange	0.655	0.655	0.655	0.655
		Mudline	Outer Flange	0.665	0.665	0.660	0.663
			Web	0.460	0.465	0.460	0.462
			Inner Flange	0.650	0.655	0.650	0.652
9	7+20	Waterline	Outer Flange	0.530	0.520	0.530	0.527
			Web	0.365	0.360	0.365	0.363
			Inner Flange	0.535	0.535	0.535	0.535
		Mid-Water	Outer Flange	0.520	0.520	0.525	0.522
			Web	0.340	0.340	0.340	0.340
			Inner Flange	0.520	0.525	0.530	0.525
		Mudline	Outer Flange	0.535	0.520	0.525	0.527
			Web	0.360	0.365	0.365	0.363
			Inner Flange	0.500	0.500	0.500	0.500
10	1+75	Waterline	Outer Flange	0.385	0.390	0.385	0.387
			Web	0.380	0.385	0.375	0.380
			Inner Flange	0.405	0.410	0.410	0.408
			HZ	0.785	0.775	0.775	0.778
		Mid-Water	Outer Flange	0.395	0.395	0.390	0.393
			Web	0.385	0.385	0.385	0.385
			Inner Flange	0.405	0.405	0.405	0.405
			HZ	0.815	0.815	0.815	0.815
		Mudline	Outer Flange	0.390	0.395	0.395	0.393
			Web	0.385	0.385	0.385	0.385
			Inner Flange	0.405	0.405	0.400	0.403
			HZ	0.785	0.785	0.790	0.787
10	4+25	Waterline	Outer Flange	0.360	0.350	0.360	0.357
			Web	0.350	0.355	0.360	0.355
			Inner Flange	0.360	0.355	0.355	0.357

			HZ	0.1200	0.195	0.1205	0.145
		Mid-Water	Outer Flange	0.355	0.350	0.350	0.352
			Web	0.350	0.350	0.355	0.352
			Inner Flange	0.350	0.350	0.35	0.350
			HZ	1.185	1.185	1.18	1.183
		Mudline	Outer Flange	0.350	0.355	0.355	0.353
			Web	0.360	0.360	0.355	0.358
			Inner Flange	0.355	0.355	0.355	0.355
			HZ	1.195	1.190	1.190	1.192
11	1+50	Waterline	Outer Flange	0.235	0.240	0.240	0.238
			Web	0.260	0.245	0.250	0.252
			Inner Flange	0.330	0.330	0.325	0.328
		Mid-Water	Outer Flange	0.285	0.285	0.280	0.283
			Web	0.250	0.240	0.235	0.242
			Inner Flange	0.275	0.270	0.27	0.272
		Mudline	Outer Flange	0.315	0.320	0.320	0.318
			Web	0.375	0.375	0.380	0.377
			Inner Flange	0.365	0.360	0.360	0.362
11	6+25	Waterline	Outer Flange	0.350	0.350	0.350	0.350
			Web	0.355	0.360	0.360	0.358
			Inner Flange	0.380	0.380	0.385	0.382
		Mid-Water	Outer Flange	0.370	0.370	0.370	0.370
			Web	0.365	0.365	0.365	0.365
			Inner Flange	0.365	0.390	0.39	0.382
		Mudline	Outer Flange	0.365	0.365	0.360	0.363
			Web	0.335	0.335	0.335	0.335
			Inner Flange	0.345	0.345	0.340	0.343

4.16 Tie rod Inspection Results

A tie rod investigation was performed at (10) locations listed in the table below. The locations were selected based on levels of observed bulkhead deterioration and to assess a representative sample of the tie rods throughout the island. The results of the investigation can be found in **Table 3** below.

Table 3: Tie Rod Inspection Summary

Section	Station	Original Diameter (in)	Condition	Comments
3	0+65	2-3/8	Moderate	Limited to tactile inspection. Moderate corrosion of outer layer, but

				no significant loss of section felt, uniform circumferential surface.
4	6+65	1-1/8	Moderate	Moderate corrosion of outer layer, uniform circumferential surface.
5	0+05 E	1-1/8	Severe	Concrete encasing tie rod crumbles, loose to the touch. Significant necking at 20-degree bend in rod, heavy corrosion.
5	2+75 E	7/8	Failed	West Bank, 100% section loss of tie rod, heavy corrosion on remaining section.
5	1+00 W	1	Moderate	Concrete encasing tie rod crumbles, loose to the touch. Moderate corrosion on surface.
6	8+40	1-1/2	Minor	Limited to tactile inspection. Smooth steel with no significant corrosion felt.
7	0+70	2	Minor	Tie rod in like-new condition, painted coating still intact.
8	7+00	5/8	Minor	3-strand steel rod encasement (3 x 5/8" dia.) encased in grease, fabric, and outer PVC layer; no significant deterioration of the steel observed.
9	5+50	5	Moderate	Minimal intermittent rust staining of the steel surface.
11	1+70	2	Minor	Expected age of tie rod is less than 5 years, no significant deterioration observed.

5 ASSESSMENT

The amount of time a structure is expected to meet its original functional intent under assumed design environmental and loading conditions is referred to as its design life. The actual time extent that a structure remains in use is referred to as its service life, which is typically greater than the design life. The typical service life of a waterfront structure is approximately 40-50 years, depending on its application and construction. The service life may be extended with proper maintenance. The ratings presented below do not consider the potential for additional damage which may be caused by a severe storm event, overloading, or continued deterioration.

Based on our field observations and the criteria established in the ASCE Manual (reference **Appendix C** for ASCE Element Level Condition Ratings), the existing structures were assigned the following condition ratings, with observed deficiencies summarized below. An overall map illustrating the condition of each section can be seen in **Figure 30**.

LEGEND

SECTION	TYPE	L.F.
1	RIPRAP REVETMENT/BREAKWATER	1205
2	RIPRAP REVETMENT/BREAKWATER	1392
3	ANCHORED STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	1350
4	ANCHORED CONCRETE SHEET PILE BULKHEAD W/ CONCRETE CAP	1030
5	603 LF ANCHORED CONC. PILE/PANEL BULKHEAD W/ CONC, ROCK TOE PROTECTION, & CONC. CAP	603
5	(2) 140LF ROCK JETTIES	280
6	ANCHORED CONCRETE PILE/PANEL BULKHEAD W/ CONCRETE CAP	1550
7	ANCHORED STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	101
8	ANCHORED STEEL SHEET PILE BULKHEAD W/ CONC. CAP & BATTER PILES	1169
9	STA. 0+00 TO STA. 0+30 ANCHORED STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	30
9	STA. 0+30 TO STA. 9+39 ANCHORED STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	909
10	STA.0+00 TO STA. 4+70 STEEL COMBINATION PILE BULKHEAD W/ CONCRETE CAP	470
10	STA. 4+70 TO STA. 5+15 STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	45
11	STA. 0+00 TO STA.9+60 ANCHORED STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	960
11	STA. 0+00 TO STA. -0+66 ANCHORED STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	66
12	ANCHORED STEEL SHEET PILE & STEEL COMBI BULKHEAD BULKHEAD W/ CONCRETE CAP	290
PARCEL 15	STEEL SHEET PILE BULKHEAD W/ CONCRETE CAP	84
13	ROCK REVETMENT	1644

CUMMINS | CEDERBERG Coastal & Marine Engineering **Miami** | Fort Lauderdale | Jupiter Tallahassee | Sarasota | St. Petersburg 201 Alhambra Circle, Suite 601 Coral Gables, FL 33134 T: +1 305-741-6155 F: +1 305-974-1969 www.cumminscederberg.com	FICA SHORELINE ASSESSMENT ONE FISHER ISLAND DR. MIAMI BEACH, FLORIDA 33109	SHORELINE CONDITION	JOB NO: 30535 DRN BY: SV SHEET: FIGURE 31	DATE: 8/26/23 CHK BY: GP

Figure 31: Shoreline Condition

5.1 Section 1

Section 1 was observed to be in overall **Satisfactory** condition (Section 1 was not inspected in the 2019 report prepared by Edgewater Resources, LLC titled, *Fisher Island Coastal Structure Assessment)*. The riprap generally presented minor deterioration. Typical deficiencies observed consisted of:

- Localized slopes steeper than 1:1

5.2 Section 2

Section 2 was observed to be in overall **Fair** condition (previously rated Good in the 2019 report The riprap generally presented minor deterioration. Typical deficiencies observed consisted of:

- Localized moderate voids due to unevenly stacked stones
- Localized minor scour pockets at the base of the riprap slope
- Severe corrosion of 6 in. diameter steel outfall

5.3 Section 3

Section 3 was observed to be in overall **Poor** condition (previously rated Satisfactory in the 2019 report). The bulkhead generally presented major to severe deterioration. Typical deficiencies observed consisted of:

- Widespread major to severe corrosion of the steel sheet pile
- Localized holes in the steel sheet pile
- Moderate corrosion of the inspected tie rod
- Intermittent moderate to major spalling with exposed fully corroded reinforcement of the concrete cap above and below water, STA 0+00 – STA 3+30
- Intermittent hairline to 1/8 in. cracking of the concrete cap throughout
- Intermittent voiding at the exposed top of sheet pile, STA 3+30 – STA 13+50
- Severe spalling with 100% section loss of reinforcement at STA 13+50 concrete overhang platform

5.4 Section 4

Section 4 was observed to be in overall **Serious** condition (previously rated Fair in the 2019 report). The bulkhead generally presented moderate to major deterioration. Typical deficiencies observed consisted of:

- Widespread moderate to major spalling and cracking of the concrete sheet pile and concrete cap throughout, including typical flexural cracking in the panels

- Intermittent sinkholes observed upland of the sheet pile in line with sheet pile joints
- Intermittent sediment accumulation at the mudline in line with upland sinkholes
- Moderate corrosion of the inspected tie rod
- Visible deflection of the wall at STA 5+70 – STA 6+35
- Localized severe spalling of the bottom edge of the cap, with exposed, fully corroded reinforcement
- Sediment accumulation causing partial blockage of 14-in. diameter corrugated steel outfalls
- Significant upland settlement at western termination of bulkhead

5.5 Section 5

The **riprap jetties of Section 5** were observed to be in overall **Satisfactory** condition. The **bulkheads of Section 5** were observed to be in overall **Poor** condition; the condition rating of the bulkheads would likely be downgraded if it were not for the substantial support provided by the riprap. (Section 5 was previously rated Fair in the 2019 report). The bulkheads generally presented moderate to severe deterioration. Typical deficiencies observed consisted of:

- Widespread severe cracking and spalling of the concrete king piles with localized delamination of the pile face
- Widespread 1/8 – ½ in. cracking with localized spalling of the concrete pile cap
- 1 ft deep sinkhole at STA 0+00 east wall connection to Section 4
- Large scale void between Section 5 and Section 4 bulkheads in line with sinkhole, significant loss of fill visible through void
- Moderate to severe corrosion of the observed tie rods
- Full separation of tie rod at STA 2+75 E
- Significant bend in tie rod observed at STA 0+05 E
- Apart from the southern-most 5–7 ft of bulkhead on each side of the channel, riprap provides considerable support to the concrete panels and king piles.

5.6 Section 6

Section 6 was observed to be in overall **Poor** condition (previously rated Fair in the 2019 report). The bulkhead generally presented major to severe deterioration. Typical deficiencies observed consisted of:

- Widespread failure of the concrete pile encasements within the tidal zone
- Localized full height delamination of the pile encasements on the offshore face
- Widespread 1/8 in. wide horizontal and diagonal cracking of the concrete panels

- Intermittent gaps between the piles and panels 2-4 in. wide with intermittent panel to panel gaps up to 2 in. wide with up to 12 in. of penetration
- Intermittent evidence of freshwater intrusion at the pile and panel interface

5.7 Section 7

Section 7 was observed to be in overall **Poor** condition (previously rated Satisfactory in the 2019 report). The bulkhead generally presented major to severe deterioration. Typical deficiencies observed consisted of:

- Widespread severe corrosion of the steel sheet pile
- 32-in. long by 16-in. wide by 21-in. deep sinkhole at STA 0+50
- 6-12 in. of sediment accumulation at joint between Section 7 and Section 8 bulkheads

5.8 Section 8

Section 8 was observed to be in overall **Fair** condition (previously rated Good in the 2019 report). The bulkhead generally presented minor to moderate deterioration. Typical deficiencies observed consisted of:

- Widespread minor corrosion and rust staining with the top 3 ft of the steel sheet pile
- Intermittent moderate to major corrosion of the installed jet filters
- Intermittent upland sinkholes
- Widespread 1/8 in. cracking with localized major spalling from STA 6+64 to Section 7
- Localized hairline to 1/8 in. cracking throughout
- Intermittent upland sinkholes throughout

5.9 Section 9

Section 9 was observed to be in overall **Fair** condition, except for the bulkhead from STA 0+00 to STA 0+30 which was observed to be in overall **Poor** condition. (Section 9 was previously rated Fair in the 2019 report). The bulkhead generally presented minor to moderate deterioration. Typical deficiencies observed consisted of:

- Widespread severe corrosion of the steel sheet pile from STA 0+00 – STA 0+30
- 18 in. high 4 in. wide void exhibits fill loss at the mudline of the Section 10 connection, STA 0+00
- Freshwater intrusion at STA 0+30 sheet pile connection

- Intermittent minor corrosion and rust staining throughout the top 3 ft of the sheet pile, STA 0+30 – STA 0+85 and STA 4+64 – STA 9+36
- Widespread moderate corrosion within the top 3 ft of the sheet pile, STA 0+85 – STA 4+64
- Minor degradation of the installed anodes
- Localized moderate to major corrosion of the installed jet filters, STA 4+64 – STA 9+36
- Moderate corrosion of the steel plates and hex bolts securing the tie rods, STA 0+85 – STA 4+64
- Intermittent cracking up to 1/8 in. with intermittent rust staining of the concrete pile cap from STA 0+35 – STA 9+36
- Intermittent minor to moderate erosion spalls of the concrete cap, STA 0+30 – STA 0+85
- Intermittent moderate erosion spalls on the concrete cap up to 5 ft long and 6 in. deep from STA 0+85 – STA 3+00
- Major spall of the concrete cap at STA 4+64

5.10 Section 10

Section 10 was observed to be in overall **Satisfactory** condition from STA 0+00 – STA 4+70 and in overall **Poor** condition from STA 4+70 to STA 5+15. (The Section 10 bulkhead was previously rated Poor in the 2019 report but has since been replaced). The bulkhead generally presented minor to moderate deterioration. Typical deficiencies observed consisted of:

- Intermittent minor corrosion of the HZ1080M piles
- Intermittent minor rust staining within the top 3 ft of the combi-wall sheet pile
- Intermittent minor transverse hairline shrinkage cracks of the concrete cap within the basin
- Widespread severe corrosion of the sheet pile, STA 4+70 – STA 5+15
- Complete destruction of the concrete cap, STA 4+70 – 4+90
- Widespread severe cracking and spalling of the concrete cap, STA 4+90 – STA 5+15
- Seabed void in sheets at connection between old and new walls, corner STA 4+70

5.11 Section 11

Section 11 was observed to be in overall **Fair** condition. (The Section 11 bulkhead was previously rated Poor in the 2019 report but has since been replaced). The bulkhead generally presented minor to moderate deterioration. Typical deficiencies observed consisted of:

- Widespread major to severe spalling of the concrete cap, STA 0+84 – STA 5+80
- Minor to moderate delamination with widespread discoloration and erosion of the concrete cap finish, STA 0+00 – STA 0+84
- Localized moderate spalling of the bottom edge of the concrete cap, STA 5+80 – STA 9+60

It should be noted that the abandoned boat ramp at the western end of Section 11 was observed to be in **Poor** condition. The ramp displayed severed cracking and settlement, with corrosion staining.

5.12 Section 12

Section 12 was observed to be in overall **Satisfactory** condition (previously rated Good in the 2019 report). The bulkhead generally presented minor to moderate deterioration. Typical deficiencies observed consisted of:

- 1 ft wide 1 ft deep sinkhole at STA 2+90 termination
- Evidence of freshwater intrusion along full height of STA 2+90 termination joint
- One location of 1/4 in. wide cracking of the concrete cap at STA 0+00
- Intermittent transverse hairline to 1/8 in. wide cracking of the concrete cap
- Localized minor abrasion spalls on the edges of the concrete cap

5.13 Parcel 15

Parcel 15 was observed to be in overall **Poor** condition (previously rated Poor in the 2019 report). The bulkhead generally presented moderate to severe deterioration. Typical deficiencies observed consisted of:

- Widespread severe corrosion of the steel sheet pile
- Voiding at the STA 0+84 termination with 1.5 ft of penetration
- Widespread severe spalling of the concrete cap below water and on the western wall
- Intermittent severe spalling with widespread moderate erosion and spalling on the remainder of the concrete cap
- Upland depression exhibiting significant ponding

5.14 Section 13

Section 13 was observed to be in overall **Fair** condition (previously rated Satisfactory in the 2019 report). The riprap generally presented minor to moderate deterioration. Typical deficiencies observed consisted of:

- Localized displacement of the stones leading to steeper slopes above water and shallower slopes with scattered stones below water from STA 0+00 – STA 8+90
- Localized areas of minor upland soil erosion

6 CONCLUSIONS AND RECOMMENDATIONS

Overall, the inspected shoreline sections are of mixed conditions, ranging from Satisfactory to Serious. Replacement is recommended for all marine structures in Serious and Poor condition, while monitoring and repairs are recommended for the remaining elements. If no remedial work is performed, routine inspections should be performed at the following intervals, or after a significant storm event: 1 year (Serious), 2 years (Poor), 10 years (Fair).

Bulkhead and riprap revetment failure could impact adjacent landscaping, hardscaping, structures, and utilities. When assigning priorities for bulkhead and riprap revetment replacement, consideration was given to condition, existing construction, estimated remaining service life, and upland elements potentially affected by bulkhead and/or revetment failure. Reference **Figure 31** below for a map depicting shoreline replacement priority, estimated remaining service life, and Rough Order of Magnitude cost of replacement.

The northeast tip of Fisher Island (Sections 12 and 13) is part of the City of Miami Beach, whereas the remainder of the island is part of Unincorporated Miami-Dade County. In accordance with the City of Miami Beach ordinance for anticipated sea level rise, bulkheads requiring replacement or significant repairs must be increased in elevation to +5.7 ft NAVD88. Unincorporated Maimi-Dade County does not have guidance relative to minimum bulkhead elevation. However, utilizing the current Southeast Florida Regional Compact Climate Change recommendation for minimum bulkhead elevation +6.0 ft NAVD88 is encouraged. To limit runoff into tidal waters, Miami-Dade County also stipulates that the upland grade behind newly installed bulkhead shall be 6 inches lower than the top elevation of the bulkhead. It should be noted that a bulkhead elevation of +6.0 ft NAVD88 is what is currently being utilized at the Fisher Island Resident Marina bulkhead reconstruction. Cummins Cederberg is available to evaluate potential sea level rise impacts on all Fisher Island properties.

LEGEND

SECTION	SCOPE	ESTIMATED REMAINING SERVICE LIFE	OPC REPLACEMENT COST
1	ALL	>40 YEARS	N/A
2	ALL	>40 YEARS	N/A
3	ALL	<5 YEARS	$5,500,000 - $5,800,000
4	ALL	1-3 YEARS	$7,000,000 - $7,300,000
5	ALL BULKHEAD	1-3 YEARS	$3,000,000 - $3,150,000
5	(2) 140 LF RIPRAP JETTIES	>40 YEARS	N/A
6	ALL	<5 YEARS	$7,350,000 - $7,750,000
7	ALL	<5 YEARS	$500,000 - $520,000
8	ALL	>30 YEARS	N/A
9	STA 0+00 - STA 0+30	<5 YEARS	$160,000 - $165,000
9	STA 0+30 - STA 9+39	>30 YEARS	N/A
10	STA 0+00 - STA 4+70	>40 YEARS	N/A
10	STA 4+70 - STA 5+15	<5 YEARS	$240,000 - $250,000
11	STA 0+84 - STA 9+60	>40 YEARS	N/A
11	STA -0+66 - STA 0+84	5-10 YEARS	$790,000 - $820,000
12	ALL	>40 YEARS	N/A
PARCEL 15	ALL	<5 YEARS	$440,000 - $460,000
13	ALL	>40 YEARS	N/A

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SHORELINE REPLACEMENT PRIORITY

JOB NO: 30535	DATE: 8/26/23
DRN BY: SV	CHK BY: GP
SHEET: FIGURE 32	

Figure 32: Shoreline Replacement Priority

6.1 Priority Recommendations

The bulkheads rated Serious and Poor are nearing the end of their service lives and planning should begin for replacement of those structures. Opinion of Probable Cost for Recommended Bulkhead Replacement is presented in Table 4 below. Due to the logistics of construction on Fisher Island, new bulkheads typically range in cost from $2,500 – $4,100 per linear foot, depending on scope and application (this is roughly 25 - 35% more than mainland costs). Replacement Costs outlined below also include 10% for soft costs (engineering and permitting), as well as up to 20% to account for mitigation and coral relocation efforts (corals were observed in the majority of the section inspected). All opinions of probable costs provided in this report are based on current market prices for materials and labor and can fluctuate with time.

Table 4: Opinion of Probable Cost for Recommended Bulkhead Replacement

Section	Approx. Linear Footage	Replacement Cost	Priority
Section 3	1,350	$5 - $5.8M	High – Replace within 5 Years
Section 4	1,030	$7 - $7.3M	Urgent – Replace within 1 - 3 Years
Section 5	600	$3.0 - $3.15M	Urgent – Replace within 1 - 3 Years
Section 6	1,550	$7.35 - $7.75M	High – Replace within 5 Years
Section 7	100	$500 – 520k	High – Replace within 5 Years
Section 9 (STA 0+00 to STA 0+30)	30	$160 - $165k	High – Replace within 5 Years
Section 10 (STA 4+70 to STA 5+15)	45	$240 - $250k	High – Replace within 5 Years
Parcel 15	84	$440 - $460k	High – Replace within 5 Years

6.1.1 Section 3

The steel sheet pile exhibits severe corrosion throughout with severe pitting and multiple full section holes. The concrete cap was of a newer construction and the inspected tie rod had no significant section loss observed. The tie rod is assumed to be representative of tie rods in the region, but it is possible that the other tie rods supporting the bulkhead vary in condition. Based on the observed deterioration, Section 3 was assigned a condition rating of Poor, with an estimated remaining service life of 5 years . The corrosion of the wall is expected to continue to develop leading to more locations of full section holes and eventually sinkholes and settlement of the upland pavers. Due the Poor condition rating as well as the upland elements potentially affected by bulkhead failure, Section 3 is a **High replacement priority and replacement is recommended within the next 5 years.**

6.1.2 Section 4

The concrete sheet pile wall exhibits widespread moderate to severe deterioration of the concrete elements. Based on the observed deterioration, the bulkhead has an estimated remaining service life of 1 to 3 years. The observed flexural failure of the panels is a precursor to collapse in those localized areas. The transverse separation of the panels indicates damage to the tongue-and-groove sheet pile mechanism, which translates to a lack of continuity in the wall. Local panel failures are more likely to occur without the support of adjacent panels through continuity. Due to the Serious condition rating, the propensity for localized bulkhead failure, as well as the upland elements potentially affected by bulkhead failure, Section 4 is an **Urgent replacement priority and replacement is recommended within the next 1 to 3 years.**

6.1.3 Section 5

Section 5 is stabilized with riprap over 90% of the bulkhead up to 2-4 ft below the cap. Given the recent bulkhead collapse in the Resident marina (same construction and age), localized failures are likely to occur in this section. The inspected tie rods showed localized failures and severe deterioration. The loss of fill at the STA 0+00 E sinkhole is significant. Based on observed conditions at the mudline, loss of fill is expected to accelerate. The remaining service life of the bulkhead is estimated at 1 to 3 years, primarily due to the observed deteriorated condition of the concrete panels, piles, and tie rods. The southern extents of the bulkheads do not have lateral riprap support and are susceptible to sudden collapse. Section 5 bulkhead is an **Urgent replacement priority and replacement is recommended within the next 1 to 3 years.**

6.1.4 Section 6

Section 6 exhibits widespread moderate deterioration of the concrete elements but has undergone significant rehabilitation efforts. The rehabilitation of the piles and the concrete cap ensure sufficient support of the bulkhead. Significant undermining repairs have successfully prevented fill loss at the toe of the panel. The washout of the pile encasements limits protection of the panel-to-panel interface and localized fill loss at sinkholes coincidental with panel joints is expected to continue. The concrete panels exhibit widespread moderate cracking and will ultimately limit the effective service life of the structure. The expected service life of the structure is predicted to be 10-15 years, with regular sinkholes and degradation of the upland fill occurring more frequently as time goes on. The replacement of the bulkhead can be prioritized as budget allows to prevent degradation of the upland fill. Due the Poor condition rating, the propensity for sudden bulkhead collapse, as well as the upland elements potentially affected by bulkhead failure, Section 6 is a **High replacement priority and replacement is recommended within the next 5 years.**

6.1.5 Section 7

Section 7 is comprised of significantly aged severely corroded steel sheet pile with a recently replaced concrete cap and tieback systems. The recently replaced elements are likely to prevent sudden collapse of the structure. The sheet pile has an estimated 5-10 years of service life remaining. The deterioration is expected to progress until full section holes develop leading to fill loss and upland sinkholes. During the inspection no full section holes were observed but a significant sinkhole was present at the corner, likely losing fill at the angled sheet pile joint. Due the Poor condition rating as well as the upland elements potentially affected by bulkhead failure, Section 7 is a **High replacement priority and replacement is recommended within the next 5 years.**

6.1.6 Section 9 (STA 0+00 to STA 0+30)

The eastern-most 30 linear ft of Section 9 bulkhead was found to have severely corroded steel sheet piles. The existing bulkhead construction in this location was not found in our historical document search and its age is unknown. The cap was replaced during 2012 and the tie rods were not inspected during the tie rod investigation. The estimated remaining service life is predicted at 5-10 years. Due to the Poor condition rating, the location adjacent to the Contractor Basin (Section 10), and because this area is often utilized to dock commercial vessels, Section 9 STA 0+00 to STA 0+30 is a **High replacement priority and replacement is recommended within the next 5 years.**

6.1.7 Section 10 (STA 4+70 to STA 5+15)

The 45 linear ft of steel sheet pile to the northeast of the basin exhibits severe corrosion throughout and partial destruction of the cap. Less than 5 years of service life is predicted to remain. Due to the Poor condition rating, the location adjacent to the Contractor Basin (Section 10), Section 10 STA 4+70 to STA 5+15 is a **High replacement priority and replacement is recommended within the next 5 years.**

6.1.8 Parcel 15

The 84 linear ft of steel sheet pile extending past the edge of the Section 12 basin exhibits severe corrosion throughout and widespread spalling of the concrete cap. Evidence of fill loss was observed at the termination joints at the ends of the bulkhead. The depression central to the upland soil further indicates significant fill loss through the structure. Parcel 15 is a **High replacement priority and replacement is recommended within the next 5 years.**

6.2 Maintenance Recommendations

The sections listed below all have an estimated remaining service life of 10 years or more. The recommendations below include recommended maintenance actions and inspection periods of the structure as recommended by ASCE.

6.2.1 Section 1

The riprap within Section 1 had minimal deterioration observed. It should be reinspected within 5 years to monitor the shifting of the stones due to vessel traffic and storm activity. Reinspection is recommended within 5 years.

6.2.2 Section 2

The outfall within Section 2 should be considered for replacement. The riprap should be reinspected within 5 years to monitor the shifting of the stones due to vessel traffic and storm activity. Reinspection is recommended within 5 years.

6.2.3 Section 8

The Section 8 bulkhead elements were constructed in 2012 and exhibit expected levels of deterioration for their age. The anodes showed signs of degradation and should be considered for replacement. Typically, bulk anodes are installed below MLW to ensure an active corrosion gradient. Currently, they are installed well above MHW, and minor corrosion of the sheet pile was observed. It is recommended that the new anodes be installed below MLW as a maintenance action. Reinspection is recommended within 4 years.

6.2.4 Section 9

The bulkhead within Section 9 (excluding STA 0+00 to STA 0+30) is mixed in age including construction from 2007 rehabilitated in 2012 and new construction in 2012. Localized major corrosion of the jet filters was observed likely due to an insufficient corrosion gradient between the filter and the anode. It is recommended that the replacement anodes be installed below MLW. Reinspection is recommended within 4 years.

6.2.5 Section 10

The combi-wall in Section 10 was replaced in 2021. The observed deterioration was minor and expected of structures of this age. Reinspection is recommended within 5 years.

6.2.6 Section 11

The steel sheet pile wall was installed in approximately 2021. The concrete cap was also replaced and exhibits widespread major to severe spalling from STA 0+84 – STA 5+80, as a result of vessel impact damage. Concrete repairs to the cap are recommended in conjunction with reinspection in 5 years to monitor the performance of the repairs. Replacement or repurposing of the concrete boat ramp is recommended, along with replacement of the associated bulkhead.

6.2.7 Section 12

The steel combi-wall was installed in approximately 2018. Only minor deterioration of the newly installed steel or concrete was observed. The sheet pile interface at the STA 2+90 exhibits observed loss of fill and signs of freshwater intrusion. If the adjacent bulkhead is not planned for replacement, the joint should be sealed to prevent further loss of fill (note: the adjacent Parcel 15 bulkhead replacement design is currently being conducted by Cummins Cederberg). Reinspection is recommended within 5 years.

6.2.8 Section 13

The Section 13 riprap is exposed to significant wave action during storm events and moderate shifting of the stones from STA 0+00 – STA 8+90 was observed. No shifting of the larger diameter stones from STA 8+90 to the eastern termination was observed. It is recommended that the shoreline continue to be monitored for degradation. Once significant erosion of the upland soil is observed, installation of new large diameter riprap from STA 0+00 – STA 8+90 is recommended. Reinspection is recommended within 5 years.

6.3 Closing Remarks

Any improvements to waterfront structures will need to meet strict regulatory permitting requirements from local, state, and federal agencies prior to construction. Cummins Cederberg is available to assist with the following services:

- Evaluation of potential sea level rise impacts to the property and waterfront structures
- Conceptual project planning, marine engineering, and construction drawings for shoreline rehabilitation
- Environmental permitting services and planning
- Construction administration, including project bidding assistance and construction inspections

The assessment and recommendations presented herein are based on the data obtained from the field observations. This report may not account for unseen variations that may exist in the current conditions. The services performed by Cummins Cederberg are consistent with the degree of care and skill ordinarily exercised by, and consistent with, the standards of the engineering profession practicing at the same time, under similar circumstances, and in a similar location as the Project. No other warranty, expressed or implied, is herewith made.

Cummins Cederberg appreciates the opportunity to assist with the marine engineering aspects of the Fisher Island shoreline. If there are any questions or concerns regarding our observations or recommendations, please do not hesitate to contact us.

7 REFERENCES

1. NOAA Tides & Currents: Virginia Key, Biscayne Bay, FL – Station ID: 8723214 (April 18, 2023), https://tidesandcurrents.noaa.gov/stationhome.html?id=8723214
2. Waterfront Facilities Inspection and Assessment Manual, American Society of Civil Engineers, 2015.
3. Underwater Inspections: Standard Practice Manual, American Society of Civil Engineers, 2001.

Appendix A – ASCE Inspection Levels of Effort

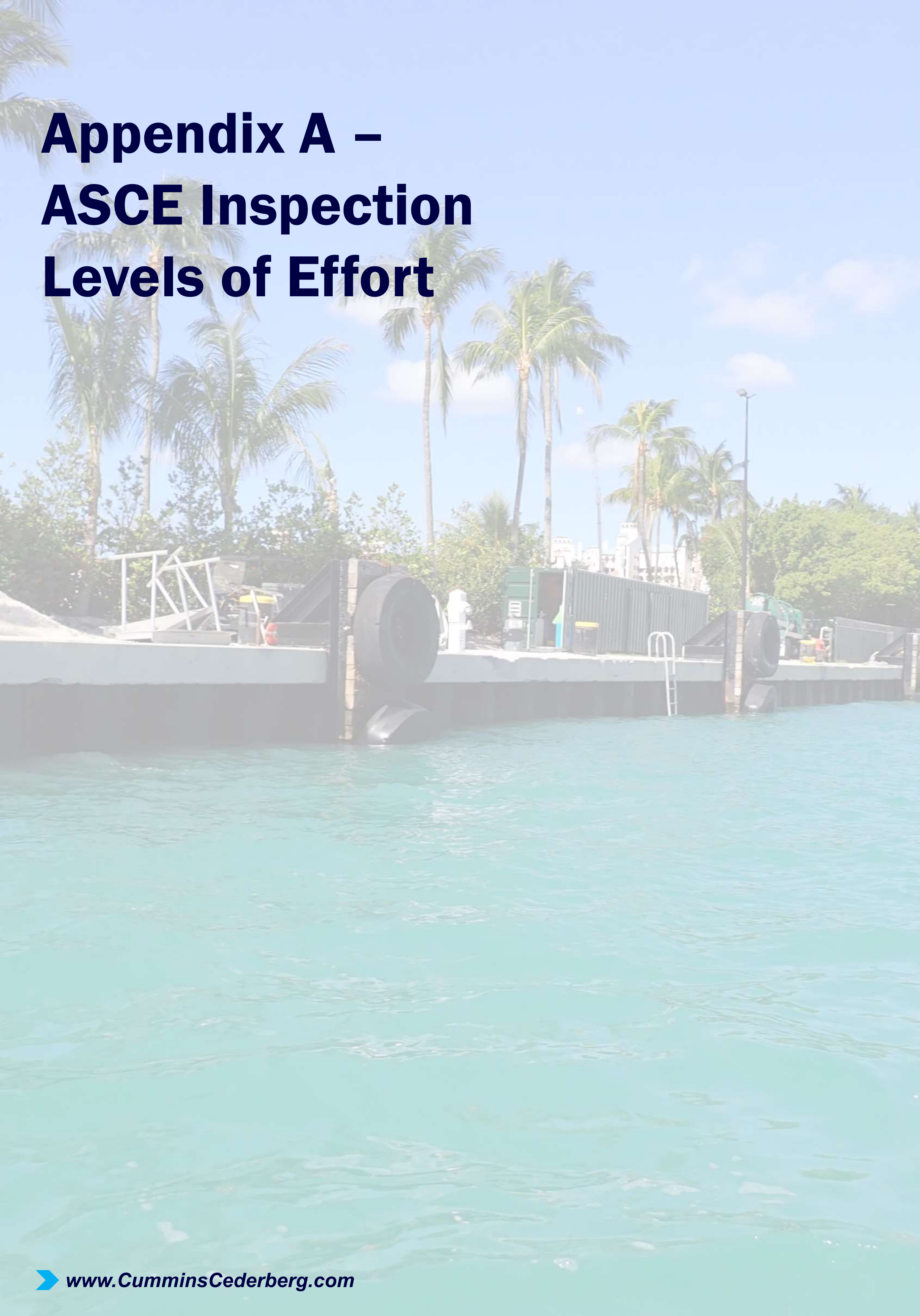
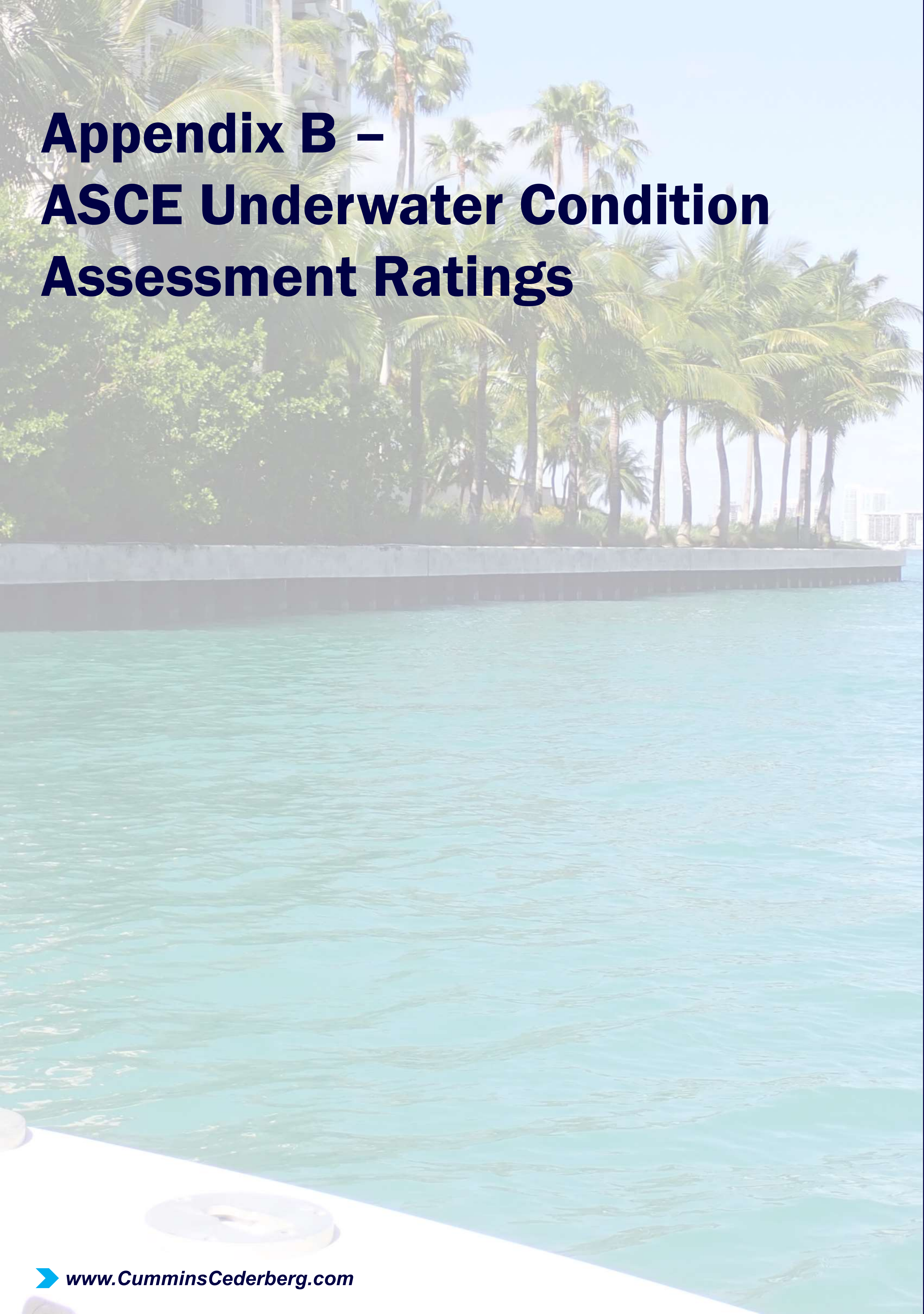


Table 1: Definition of Inspection Levels of Effort

Level	Definition
I	Includes a close visual examination above and underwater or a tactile examination using large sweeping motions of the hands where visibility is limited underwater. Although the Level I effort is often referred to as a “swim by” inspection, it must be detailed enough to detect obvious major damage or deterioration due to overstress or other severe deterioration. It should confirm the continuity of the full length of all members and system components and detect undermining or exposure of normally buried elements. A Level I effort may also include limited probing of the substructure and adjacent channel bottom.
II	A detailed inspection above and underwater that requires wrappings, coatings, corrosion, and/or marine growth to be removed from portions of the structure. Underwater marine growth removal is costly, hence, the need to base the inspection on a representative sampling of components. For piles, a 12-in. high band should be cleaned at designated locations, generally near the low waterline, at the mudline, and midway between the low waterline and the mudline. On a rectangular pile, the marine growth removal should include at least three sides; on an octagonal pile, at least six sides; and on a round pile, at least three-fourths of the perimeter. On large-diameter piles, 3 ft or greater, 1 ft × 1 ft areas should be cleaned at four locations approximately equally spaced around the perimeter, at each elevation. On large solid faced elements, such as retaining structures, 1 ft × 1 ft areas should be cleaned at these three elevations. The Level II effort should also focus on typical areas of weakness such as connections, attachment points, and welds. The Level II effort is intended to detect and identify damaged and deteriorated areas that may be hidden by surface biofouling, coating, or corrosion, or that which may not be readily accessible for a Level I inspection effort. The thoroughness of marine growth removal should be governed by what is necessary to discern the condition of the underlying material. Removal of all bio-fouling

	staining is generally not required. Means and methods for the removal of bio-fouling growth are not typically defined in a scope of work. However, it may be appropriate for owners to specify particular methods based on environmental and site conditions or on concern for maintaining the integrity of coating materials. Methods may include hand scrapers or mechanical systems ranging from high pressure water blasters to barnacle busters and pressurized air bubble devices based on the principles of cavitation.
III	A detailed inspection above and underwater typically involving nondestructive or partially destructive testing conducted to detect hidden or interior damage, or to evaluate material homogeneity. Typical inspection and testing techniques include the use of ultrasonic, coring or boring, physical material sampling, and in situ hardness testing. Level III testing is generally limited to key structural areas, areas that are suspect or areas that may be representative of the structure or system.

Reference: American Society of Civil Engineers (ASCE) Manual on Engineering Practice No. 130: Waterfront Facilities Inspection and Assessment, 2015.



Appendix B – ASCE Underwater Condition Assessment Ratings

Appendix B

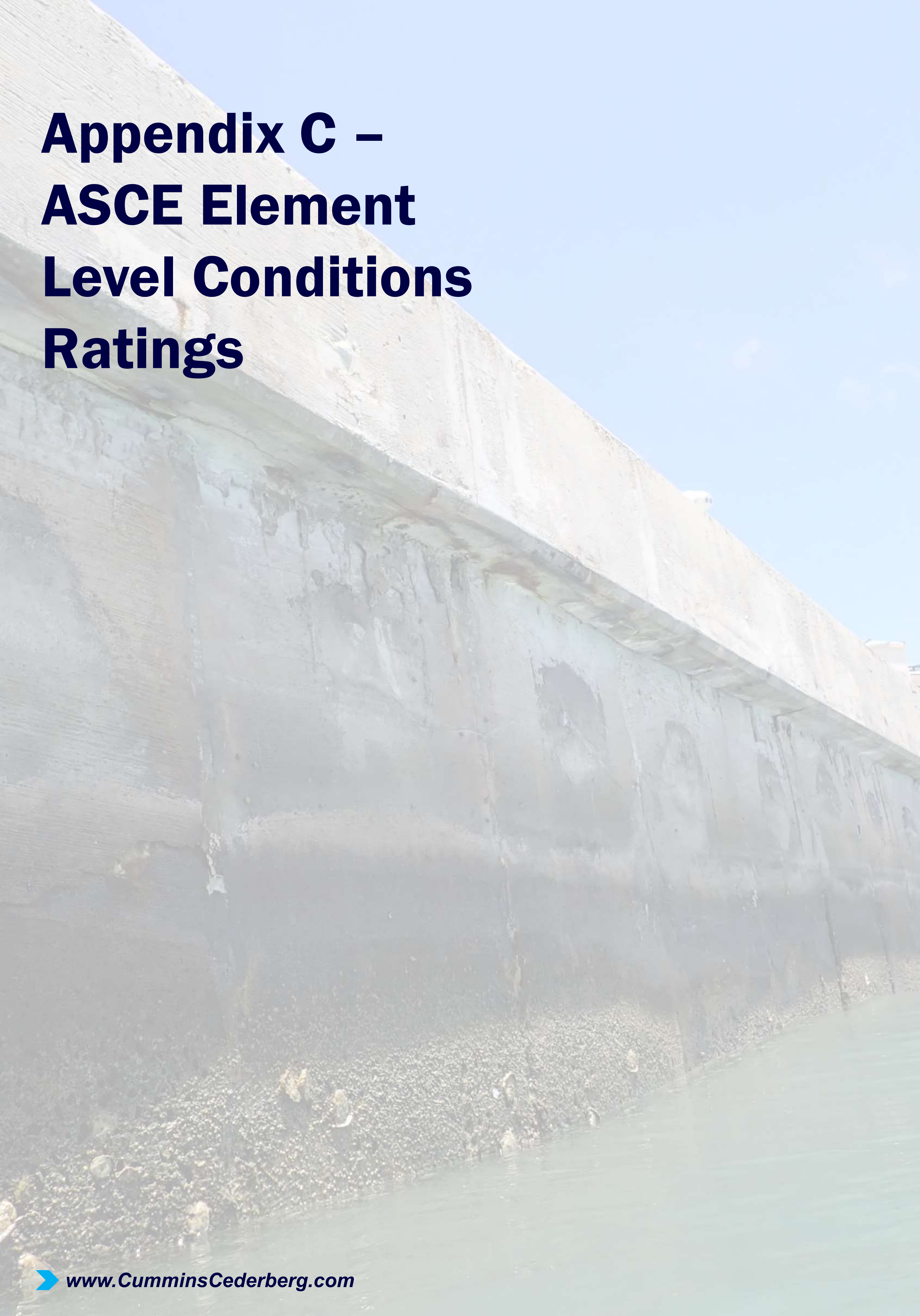
Table 2: Routine Underwater Condition Assessment Ratings[1]

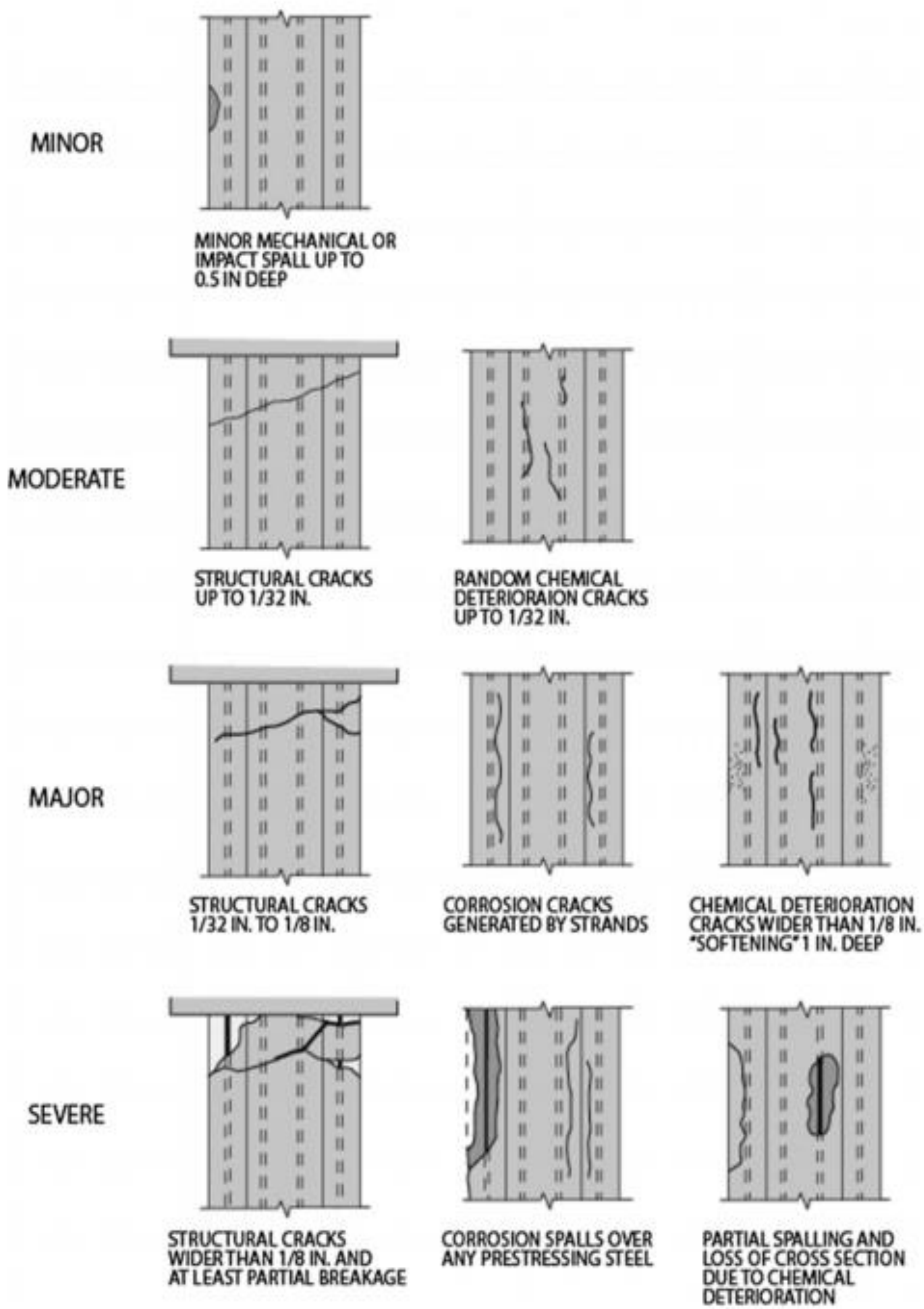
Rating		Description
6	Good	No visible damage, or only minor damage is noted. Structural elements may show very minor deterioration, but no overstressing is observed. No Repairs are required.
5	Satisfactory	Limited minor to moderate defects or deterioration are observed, but no overstressing is observed. No Repairs are required.
4	Fair	All primary structural elements are sound, but minor to moderate defects or deterioration is observed. Localized areas of moderate to advance deterioration may be present but do not significantly reduce the load-bearing capacity of the structure. Repairs are recommended, but the priority of the recommended repairs is low.
3	Poor	Advanced deterioration or overstressing is observed on widespread portions of the structure but does not significantly reduce the load-bearing capacity of the structure. Repairs may need to be carried out with moderate urgency.
2	Serious	Advanced deterioration, overstressing, or breakage may have significantly affected the load-bearing capacity of primary structural components. Local failures are possible and loading restrictions may be necessary. Repairs may need to be carried out on a high-priority basis with urgency.
1	Critical	Very advanced deterioration, overstressing, or breakage has resulted in localized failure(s) of primary structural components. More widespread failures are possible or likely to occur, and load restrictions should be implemented as necessary. Repairs may need to be carried out on a very high priority basis with strong urgency.

Reference: American Society of Civil Engineers (ASCE) Manual on Engineering Practice No. 130: Waterfront Facilities Inspection and Assessment, 2015.

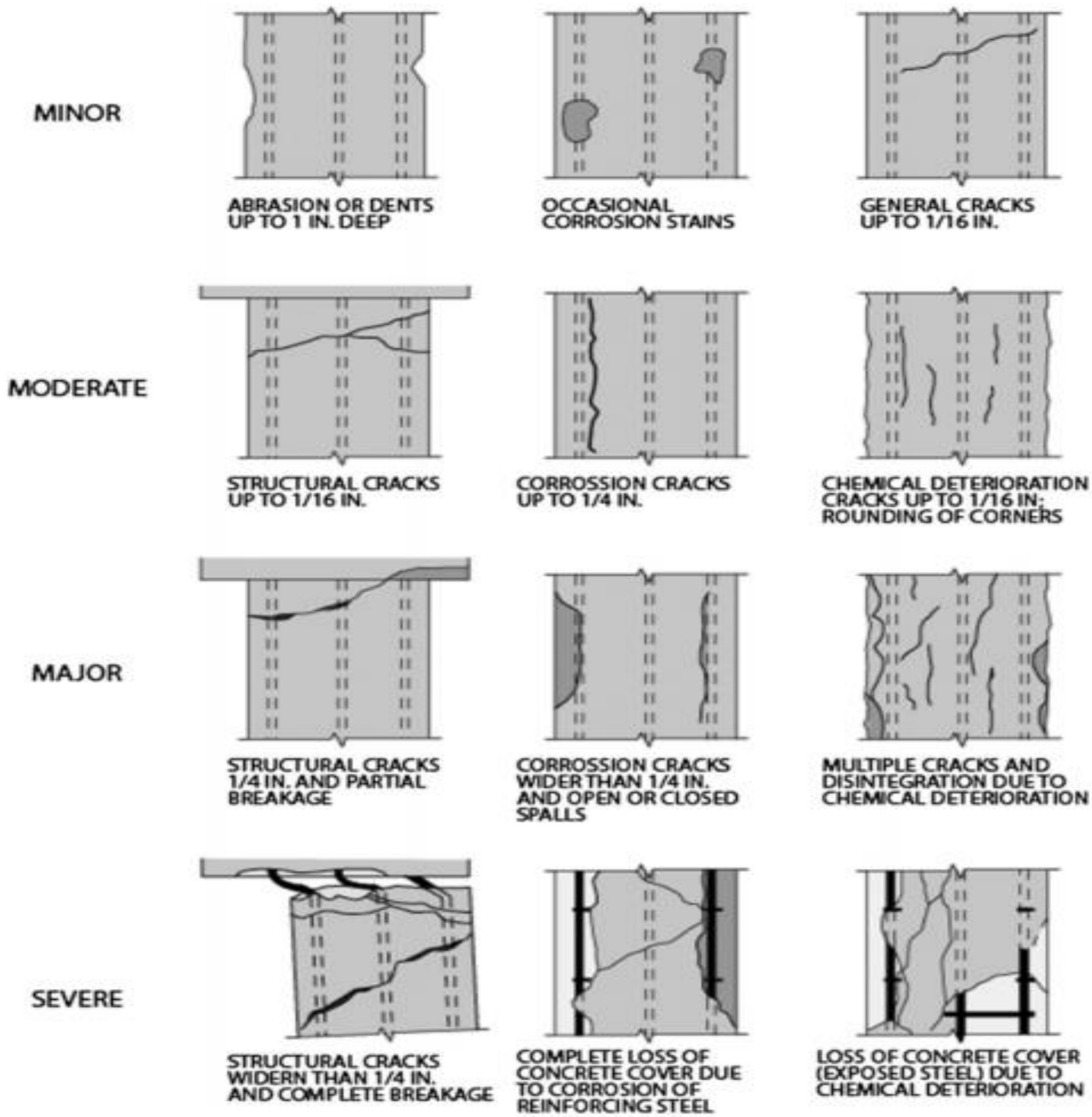
[1] Ratings are used to describe the existing structure compared with the structure when newly built. The possibility that the structure may have been designed for loads that are lower than the current standards for design should have no influence on the ratings.

Appendix C – ASCE Element Level Conditions Ratings

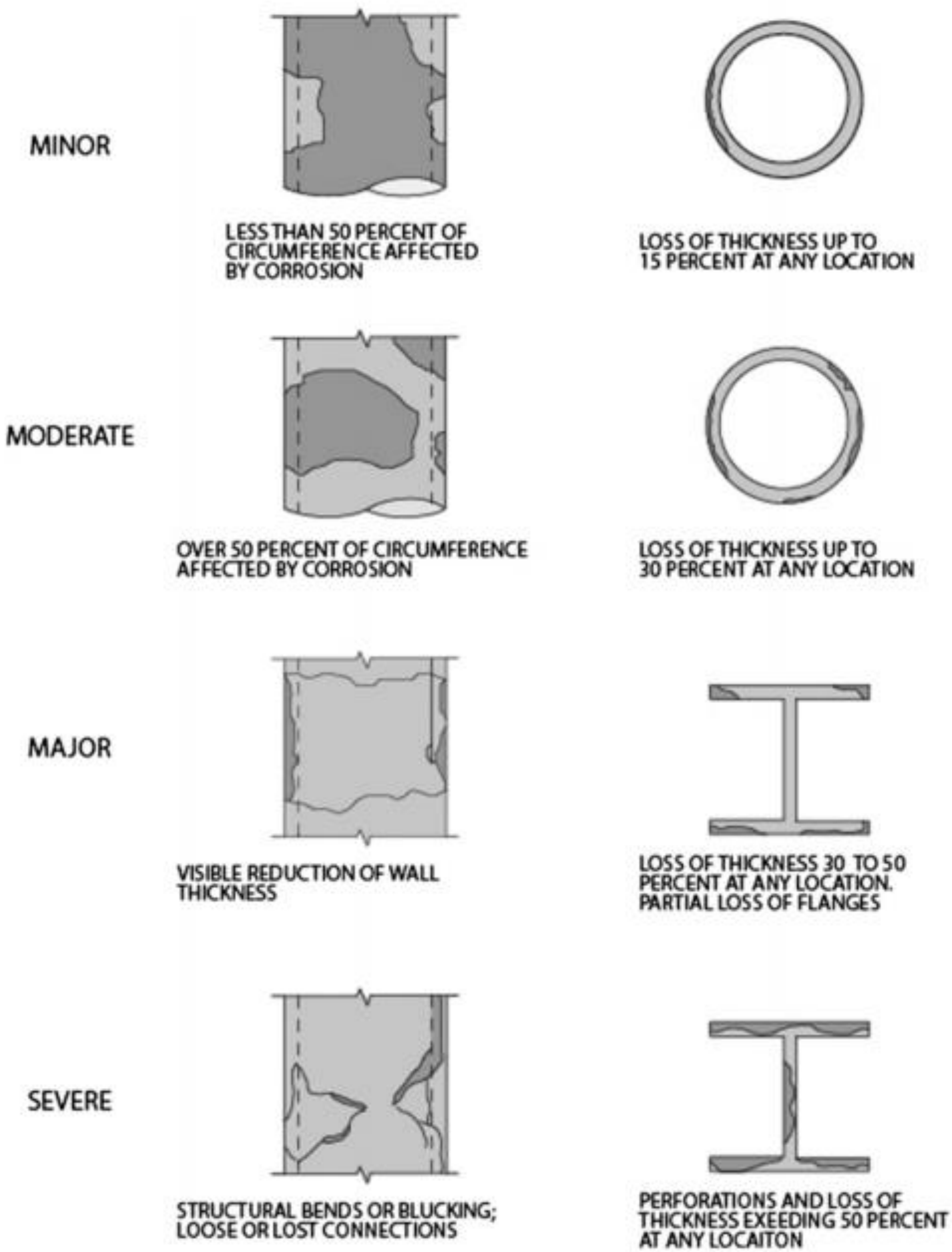




Damage Ratings for Pre-stressed concrete elements



Damage Ratings for Reinforced Concrete Elements



Damage Ratings for Steel Elements