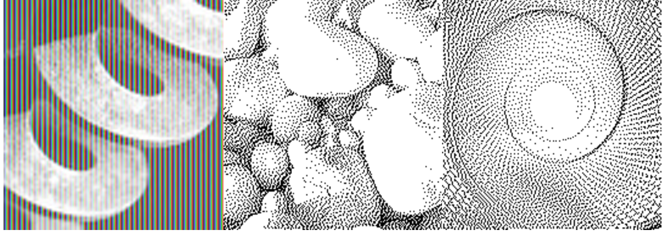

Consulting
Engineers and
Scientists

Ferry Terminal Inspection and Assessment
FINAL REPORT

Bar Harbor, Maine

Submitted to:
Town of Bar Harbor
93 Cottage Street
Bar Harbor, ME 04609

Submitted by:
GEI Consultants, Inc.
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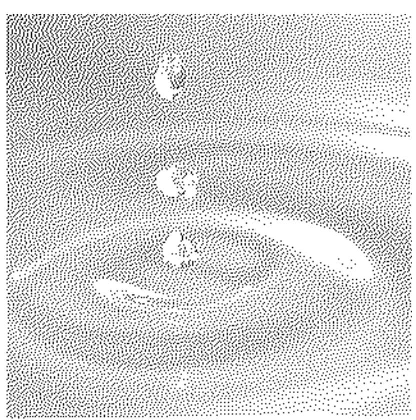
February 8, 2021
Project 2004148



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Executive Summary

In 2020, the Town of Bar Harbor retained GEI Consultants, Inc. for an inspection and assessment of the marine structures at the Ferry Terminal property located at 121 Eden Street, Bar Harbor, ME. GEI's work on the project included completion of an above and underwater inspection of the facility, hydrographic survey of the harbor in the vicinity of the site, drone-based topographic survey of the adjacent upland, structural analysis and evaluation of the existing pier structures, and development of recommendations for rehabilitation of the existing structures if they are to remain in service, as well as preparation of conceptual layouts for potential future redevelopment of the site as a recreational marina.

As part of the communication protocol to keep the Town informed of work progress, two draft reports were submitted at key project milestones. Additionally, three interim presentations were made to the Harbor Committee on 11/9/2020, 12/14/2020, and 1/11/2021. This report represents the third and final submission for the project and documents all work undertaken to date by GEI. Report submissions are summarized in the table below:

Submission	Description	Date
A	Task 2 Draft Submission (Background Information Review)	10/12/2020
B	Task 4 Draft Submission (Field Inspection)	11/13/2020
C	Final Submission	2/08/2021

The existing marine structures at the Bar Harbor Ferry Terminal were constructed c. 1956 to serve as a U.S. port for the MV Bluenose which historically provided ferry service between Yarmouth, Nova Scotia, CA, and Bar Harbor, ME. Ferry service continued at the site through 2009 but has not been operating since. In 2019, the Town acquired the property. The Town currently has an active lease with Bay Ferries Ltd. of Charlottetown, Prince Edward Island, CA to use the facility for berthing The CAT ferry. Service to Bar Harbor was proposed to begin in 2019, however the 2019 season was cancelled due to status of site facilities, and the 2020 and 2021 seasons were cancelled due to the COVID-19 pandemic. As a result, ferry service has not been active for more than 10 years.

In recent years the Town has undertaken several in depth planning efforts to evaluate potential future uses of the site. While the future use has not been definitively determined, much discussion has centered around redeveloping a facility that would serve recreational boating, potentially in combination with working waterfront, along with the piers, slips, a boat ramp, access, and services that would be required to support such uses.

The two large steel pile-supported concrete piers that currently exist at the site are now 65 years old. Throughout the lifespan of these structures, several projects have been undertaken to modify or upgrade elements to address changes in use or general deterioration. At the present time the piers are generally in a state of widespread disrepair, with conditions of elements observed by GEI during the inspection ranging from "C – Fair" to "F – Critical" (on an A-F scale). The current condition can be attributed to a range of factors, including:

<u>Normal deterioration that can be expected to occur over the life of a structure in an aggressive marine environment</u>: At 65 years old, the structure would be nearing or at the end of the design life that would typically be estimated for a similar structure. For example, the estimated lifespan of the concrete encased steel H-piles used in the construction, based on ASCE 130 guidelines, is approximately 60-75 years assuming an average rate of corrosion.

<u>Advanced deterioration related to type of construction that was used</u>: Although original construction included concrete encasement in the tidal zone (high corrosion zone), the use of steel H-piles in a corrosive marine environment will result in rapid corrosion due to the high ratio of exposed surface area to volume of steel. The use of field welds in steel H-piles within the tidal range also results in further accelerated corrosion at splice locations.

<u>Advanced deterioration due to deferred maintenance</u>: As one example, a previous evaluation performed in 2008 put forth a list of high (Phase 1), medium (Phase 2), and low (Phase 3) priority repairs with recommendations to complete the phases of repairs within a 1, 2, and 5 year timeframe, respectively. There is no evidence that these repairs were completed.

<u>Damage resulting from specific events:</u> For example, damage to the timber breasting piles on the northeast corner of the North Pier related to the fire reported to have occurred on the structure.

At this stage in the structure's lifespan, an extensive repair program would be required to keep the pier in service. In the immediate short-term, operational constraints have been recommended to limit loading on the structure during use by the CAT. These limitations will need to be reviewed with Bay Ferries operations staff and implemented. Conceptual recommendations and cost estimates for a short-term and long-term repair program are provided in this report. These repairs will need to be completed if the pier is to remain in use or be reopened for a future use. Given the significant cost of the work and the limited remaining lifespan, the Town should carefully consider the feasibility of maintaining, as opposed to replacing, the existing structure. The assessment of repair versus replacement should consider the cost-effectiveness and technical feasibility of implementing the repairs, as well as the compatibility of the existing structure with the future uses that are being considered.

To assist in consideration of potential future use as a recreational marina, GEI has also provided conceptual designs and cost estimates for three potential marina layouts at the site that include variations of selective demolition, reuse, repair, and replacement of the existing structures. Many questions will still need to be answered through detailed site investigations and design development if a marina design is selected to go forward, however the concepts provide a range of options that can be considered during early stage planning.

1. Introduction

The Town of Bar Harbor, Maine has retained GEI Consultants, Inc. for an inspection and assessment of the marine structures at the Ferry Terminal property located at 121 Eden Street, Bar Harbor, Maine. The existing facility was formerly owned by the Maine State Ferry Service and supported ferry operations from Bar Harbor, Maine to Nova Scotia, Canada. In 2009, the ferry ceased to operate. In the years following, the Town of Bar Harbor has explored a range of options for reuse or redevelopment of the facility, including multiple studies on the economic feasibility of various concepts. In 2019, the Town of Bar Harbor acquired the property. The deteriorated condition of the existing marine structures resulting from a history of deferred maintenance are known constraints to potential redevelopment of the waterfront.

The intent of the present assessment is to evaluate the current condition of existing marine infrastructure. This evaluation will provide an understanding of facility condition, serviceability, and limitations presented on future use in order to help inform the Town's next steps in planning for future waterfront redevelopment at the site.

1.1 Scope of Work

The scope of work for this study, as described in GEI's proposal dated May 29, 2020, includes the general tasks outlined below.

Contract Task	Description
1	Project Kickoff
2	Background Information Review
3	Project Coordination
4	Field Inspection
5	Structural Analysis
6	Conceptual Design of Rehabilitation Measures
7	Progress Meeting
8	Final Reporting

The field inspection included topside/above water and underwater inspection of the existing facility following the procedures of American Society of Civil Engineers (ASCE) Manuals and Reports on Engineering Practice No. 101 – Underwater Investigations (MOP 101) and also utilizes ASCE Manuals and Reports on Engineering Practice No. 130 – Waterfront Facilities Inspection and Assessment (MOP 130) as a reference. The type of inspection is classified as "Routine Inspection" as defined in MOP 130, the purpose of which is "to assess general condition, assign condition rating, and provide recommendations for future maintenance." Level I and Level II inspection were performed for the entire facility within the inspection scope and Level III inspection was performed for the piles during underwater inspection. All inspection was performed by GEI's in-house engineer-dive team and support vessel. Topside inspection was supplemented by a drone-based inspection to capture high-resolution imagery and video in a series of flights above and around the pier structures. Additional detailed discussion on inspection procedures is provided in Section 1.5.

During the inspection, a hydrographic survey was undertaken in the vicinity of the pier using GEI's support vessel with on-board GPS and single-beam sonar depth sounding equipment. The hydrographic survey was completed at high-tide over an area of approximately 13 acres including areas around and beneath the piers, adjacent to and between the north and south causeways, and extending beyond the limits of the pier approximately 150 ft in all directions. Although not specified as part of the contract, additional drone-based topographic survey data was collected of the intertidal and immediate upland property using GEI's drone with photogrammetric survey capability by flying the area at low tide. Additional detailed discussion on survey procedures is provided in Section 1.6.

1.2 Site Description

1.2.1 General Description

The Bar Harbor Ferry Terminal is located at 121 Eden Street in Bar Harbor and is identified as Lot 004 on Town of Bar Harbor Tax Map 231. The site is located at approximate Lat/Long coordinates 44°23'58.82"N, 68°13'28.41"W. The site is depicted on NOAA Chart 13323 "Bar Harbor, Mount Desert Island". The upland property has a total area of 6.78 acres, with 467 ft of frontage on Frenchman Bay (937 ft measured along Highest Annual Tide line) based on a 2019 boundary survey by CES, Inc. The facility is depicted in Fig. 1.

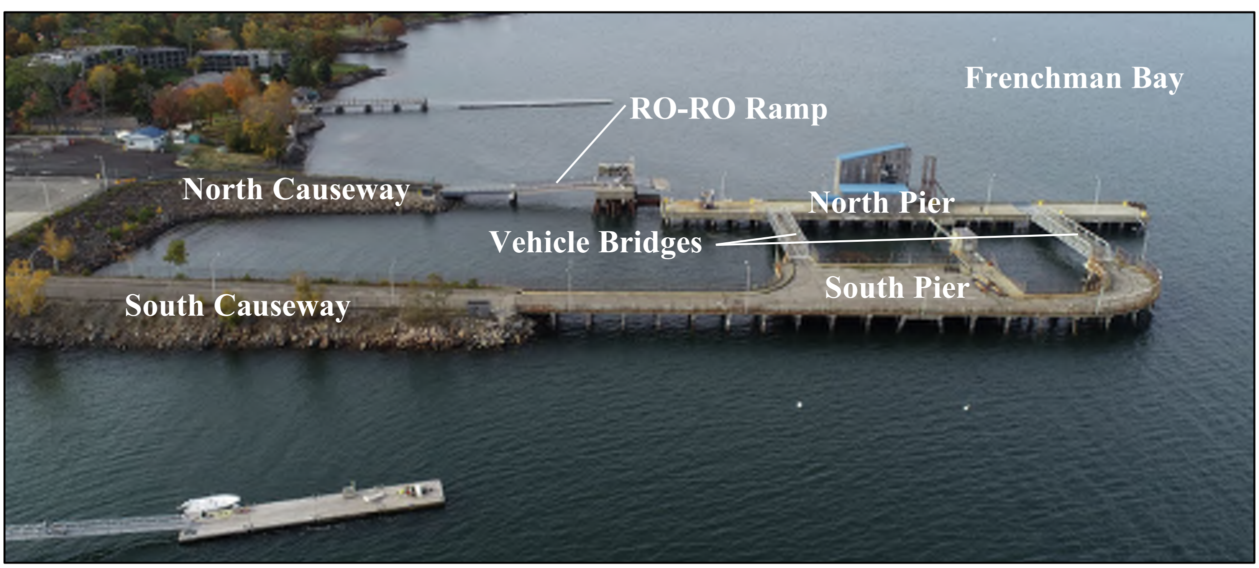

Fig. 1 - Ferry Terminal, October 2020 Drone Imagery

Marine structures at the facility include:

- North and South Approach Causeways approximately 220 ft and 350 ft long, respectively, with riprap revetment slopes.

- A South Pier that extends approximately 400 ft outshore of the Approach Causeway. The pier construction consists of the following: vertical and battered steel H-piles jacketed with concrete and timber from approximately 4 ft below low water up to the pile caps, reinforced concrete pile caps and framing, and a reinforced concrete deck slab, with asphalt wearing surface.

- A North Pier that is 380 ft long by 40 ft wide. The pier construction consists of the following: vertical and battered steel H-piles jacketed with concrete and timber from approximately 4 ft below low water up to the pile caps, integral steel pile caps and framing with concrete encasement, and a reinforced concrete deck slab. There is a submerged rubble mound and 380 ft long by 40 ft wide submerged concrete slab located beneath a portion of the pier. A timber pile cluster is located at the northeast corner of the pier.

- Two (2) steel truss bridges for vehicle access between the South and North piers which historically articulated to provide side-loading access to vessels but are currently fixed at deck level.

- Remnants of a former pedestrian access bridge located between the two vehicle bridges that provided access from the south pier to the loading building on the north pier. The pedestrian bridge is no longer present, but the pile bents that historically supported the bridge are with utility structures hanging from them.

- A dilapidated building located on the north pier that is a remnant of the former ferry service use.

- A RO-RO transfer bridge, steel pontoon barge, and pile dolphins extending from the north causeway that are owned by Bay Ferries were not included in the scope of inspection.

- Fendering and mooring hardware along the north face of the North Pier that is used by Bay Ferries vessels. These components include the UHMW faced steel marine fender panels, compressible rubber cylinder fenders, floating fenders with tire netting, steel fender piles, mooring structures, and large steel bollards on concrete bases. These components are maintained by Bay Ferries and were not included in the scope of inspection.

1.2.2 Tidal and Flood Elevations

Tidal and flood elevations for the site are summarized in Table 1 in the following datums: Mean Lower Low Water (MLLW), Mean Low Water (MLW), National Geodetic Vertical Datum of 1929 (NGVD29), and North American Vertical Datum of 1988 (NAVD88).

Table 1. Water Elevations

Elevation Reference (all elevations are in feet)		Vertical Datum			
		MLLW*	MLW	NGVD29	NAVD88
Base Flood Elevation		**+22.97**	+22.59	+17.63	+17.00
Highest Observed Water Level (February 7, 1978)		**+16.21**	+15.83	+10.87	+10.24
Stillwater Elevation	0.2% Annual Chance	**+15.67**	+15.29	+10.33	+9.70
	1% Annual Chance	**+15.27**	+14.89	+9.93	+9.30
	2% Annual Chance	**+14.77**	+14.39	+9.43	+8.80
	10% Annual Chance	**+13.77**	+13.39	+8.43	+7.80
Highest Annual Tide (HAT)		**+13.31**	+12.93	+7.97	+7.34
Mean Higher High Water (MHHW)		**+11.37**	+10.99	+6.03	+5.40
Mean High Water (MHW)		**+10.94**	+10.56	+5.60	+4.97
NAVD88		**+5.97**	+5.59	+0.63	0.00
Mean Sea Level (MSL)		**+5.67**	+5.29	+0.33	-0.30
NGVD29		**+5.34**	+4.96	0.00	-0.63
Mean Low Water (MLW)		**+0.38**	0.00	-4.96	-5.59
Mean Lower Low Water (MLLW)		**0.00**	-0.38	-5.34	-5.97
Lowest Observed Water Level (March 21, 2007)		**-2.91**	-3.29	-8.25	-8.88

*Project Datum

Site specific tidal elevations and datum conversions were derived from NOAA Tidal Station 8413320, Bar Harbor, ME. The Highest Annual Tide (HAT) elevation was taken from the Maine DEP published HAT table for 2018. Stillwater elevations were taken from the 2016 FEMA Flood Insurance Study (FIS) for Hancock County. Coastal Stillwater Elevations were taken from Transect 109 of the FEMA FIS which is located directly adjacent to the site. The Base Flood Elevation was taken from FEMA flood map number 23009C1013D which shows the site in a VE zone with Base Flood Elevation of +17 ft NAVD88. The FEMA FIS lists starting wave conditions for the 1% Annual Chance for Transect 109 as a Significant Wave Height (H_s) of 10.1 ft, and a Peak Wave Period (T_p) of 9.3 seconds.

All elevations provided in this report and on the project plans are referenced to MLLW (chart) datum unless otherwise noted.

For reference, the top of deck elevation of the existing pier is approximately +22.4 to +22.9 ft MLLW, which is 9+ ft above the Highest Annual Tide elevation, 7+ ft above the 1% annual chance stillwater elevation, and slightly lower than the Base Flood Elevation.

1.3 Review of Background Information

Extensive information has been provided on the history of the pier and the site. GEI has reviewed the documents provided by the Town, as well as additional documents and reports acquired from Prock Marine Company, Cianbro Corporation, and several other sources. A list of references reviewed is provided in Appendix A. A summary of the chronological history of marine structures is provided below.

The focus of the review and commentary contained in this section is the construction, repair, modification, and past inspection/assessment of the marine structures at the facility. Many of the documents that were reviewed relate to the upland facility and buildings onsite, utilities, or other ancillary items which may be of interest to the Town but are not pertinent to the present scope of structural assessment. Therefore, these items were not included.

It is also understood that a thorough planning process has been undertaken by the Town recently to consider future uses of the property. While these recent planning efforts are not summarized in this section, it is noted that they will be relevant to later phases of analysis and design, during which potential future uses of the facility will be considered.

1.4 Facility History

Year	Item/Description
1954	The original design plans for the terminal facility were prepared by Faye, Spofford & Thorndike Engineers and are dated 1954. A limited number of sheets from the original plans are available. (References No. 10-17).
1982	West vehicle ramp modified from historically articulating condition to fixed ramp and deck at location of ramp interface infilled with concrete slab (References No. 19 and 73)
1986	Fixed and movable ramps acting as interface between vessels and passenger walkway replaced with telescopic ramp, designed and constructed by Apex Machine Works Limited. (Reference No. 56).
1991	Approximately 180 ft of fendering system on northern face of pier replaced with fender panels and another 90 ft repaired by Reed & Reed Inc. (References No. 28, 29, 46-53).
1992	East vehicle ramp modified from historically articulating condition to fixed ramp (Reference No. 73)

Year	**Item/Description**
1995	An analysis of the pier was completed by Skarborn Engineering. A copy of this report has not been made available but is referenced in the 2008 inspection report by EastPoint Engineering (Reference No. 73).
1995	Geotechnical investigation performed along north face of pier by S.W. Cole Engineering. The exploration found loose to very dense granular soils in the top 11 to 18 ft. The granular soils were underlain by soft to very soft gray silty clay extending to the 26 to 39 ft depth. The clay was underlain by glacial till and refusal. (Reference No. 57).
1995	Approximately 90 ft of existing original fender system along north face of pier replaced with fender panels by Prock Marine Company. (References No. 8, 34, 35).
1996	Upgrades to both vehicle access ramps performed by Knowles Industrial Service Corporation, including recoating, timber replacement, and repairs to concrete pile caps. Inspection of work completed by Stone-Fleming Consulting. (References No. 4-7, 9, 32).
1996	Replacement of the east access ramp support structure completed by Skarborn Engineering and K.V. Mechanical & Welding Ltd. (Reference No. 9).
1996	Repairs to damaged passenger walkway canopy performed by Associated Builders. (Reference No. 65).
1998	Four westernmost vertical timber fenders replaced with UHMW plastic material. (Reference No. 74).
2007	The most recent underwater inspection, to our knowledge, was performed in 2007 by Marenco Engineering, Ltd. of Charlottetown, PE, CA (Reference No. 72). Key findings of this inspection related to the piles and deck include: • 40% of the piles supporting the north pier and 25% of those supporting the south pier have significant loss of section just above chart datum. • Over 90% of the piles on the north pier and about 30% of those on the south pier are assumed to have splices with full penetration welds. Once exposed to the atmosphere, all those welds are expected to deteriorate quickly.

Year	Item/Description
	• Structural repairs are recommended in the immediate future. If the rate of deterioration of the last five years continues, the piers will not be safe for use in another five years. • Underwater welding should not be considered as a repair option. • The requirement for load restrictions on the piers should be explored. • The concrete is in relatively good condition. There are approximately 80 m^2 (860 ft^2) of spalled concrete and exposed reinforcing steel which should be patched. • Before repair options are considered, the time at which the piers will be replaced, rebuilt or taken out of service should be determined. The inspection report also refers to a previous underwater inspection performed in 2001, noting that conditions have become significantly worse since the 2001 inspection. A copy of this report has not been located by GEI.
2008	A structural assessment was performed by EastPoint Engineering, Ltd. of Halifax, NS, CA (Reference No. 73), apparently as follow-up to the 2007 inspection by Marenco. The assessment noted the significant deterioration described in the 2007 Marenco inspection report, noting that "the number of piles that are virtually nonexistent due to the loss of flange and web material due to corrosion has increased at what would be considered an exponential rate since 2001." Key findings of the 2008 report include: • The structure is at the beginning of exponential loss of structural capacity of the piles that support the north pier with the south pier following in a short period of time. • Given the number of class 3 and class 2 (deteriorated) piles…it is imperative that a pile repair program be started immediately. • Loading on the eastern portion of the south pier was recommended to be restricted to ½ ton truck loading. • Until all of the north-south class 4 batter piles are repaired vessels should not berth against the structure in sustained wind conditions that exceed 50 mph.

Year	Item/Description
	A program of repairs to the supporting piles of the pier was recommended as part of this report. It is unknown at this time whether any of the recommended repairs were completed.
2009	Ferry Service ends at the facility.
2011 to 2018	Feasibility Study for the acquisition of the Bar Harbor Ferry Terminal completed by Bermello, Ajamil & Partners, Inc. (Reference No. 75). Deferred maintenance and significant repair costs are identified. Numerous additional phases of study undertaken to assess potential future uses of ferry terminal property and potential for acquisition by Town. (References No. 76-91)
2019	Property acquired by Town of Bar Harbor.
2019	Repairs to the north pier were completed by Cianbro Corp. which included upgrades to fendering on the north face of the pier, and repair of 12 piles beneath the north pier by bolting steel channel sections to H-piles where severe section loss had occurred (Reference No. 92). During a phone conversation with Pat Sughrue of Cianbro on November 5, 2020, it was also noted that in a similar timeframe Cianbro had provided a price to Bay Ferries for demolition of the pier superstructures, but the demolition did not proceed due to pricing being prohibitive.
2020	Bay Ferries was scheduled to resume ferry service to the facility in 2020, but all trips for the season were cancelled due to the ongoing COVID-19 crisis.

1.5 Inspection Methodology

GEI personnel self-performed an inspection of the Ferry Terminal facility between the dates of Monday, October 19 to Friday, October 23, 2020. Personnel onsite during the inspection included: Daniel Bannon, P.E., Andrew Cameron, E.I.T., Stephen Hennessy, E.I.T., and Daniel Pelletier, E.I.T. Weather conditions during the inspection were clear and calm.

The Routine Inspection as defined in the American Society of Civil Engineers (ASCE) Manuals and Reports on Engineering Practice No. 130 "Waterfront Facilities Inspection and Assessment" (MOP 130) and ASCE Manuals and Reports on Engineering Practice No. 101 "Underwater Investigations" (MOP 101). Level I and Level II inspection was performed for all elements within the inspection scope. Level III inspection was performed for piles during the underwater inspection of piles and for above water elements where possible. The inspection level definitions applicable to the project scope are summarized below:

- Level I Inspection: Visual and tactile inspection on 100% of structure.
- Level II Inspection: Visual and auditory inspection and measurements of topside elements as necessary. Partial growth removal and cleaning on 10% of steel piles.
- Level III Inspection: Remaining thickness measurements and electrical potential measurements on 5% of steel piles.

The topside and above water inspection consisted of visual, tactile, and auditory inspection of the concrete deck on the North Pier, concrete deck and asphalt wearing surface on the South Pier, eastern and western steel truss bridges, timber curbs and mooring structures on the North Pier, and rip rap shoreline and causeway slopes. The topside structures were accessed by foot from the pier deck. The underside of the pier deck and pile cap were visually inspected with access from the work boat at high tide and from the adjacent shoreline. Exposed portions of the pier piles were inspected from the work boat at various times throughout the tidal range. Additional topside and above water visual inspection was undertaken by performing a series of low-level drone flights around the structure to document conditions with high-resolution photographs and video. The drone-based visual inspection was performed by a FAA licensed pilot with a DJI Phantom Pro 4 drone and onboard camera.

The underwater inspection was performed by in-house engineer-divers and consisted of visual and non-destructive physical inspection of the steel H-piles and their concrete and timber jackets. Level I inspection was performed on 100% of piles and included visual and tactile inspection of the in-situ piles. Level II inspection was performed on at least 10% of the piles, which included partial cleaning of marine growth to allow for visual inspection to identify deterioration that may not have been revealed by Level I inspection. Level III inspection was performed on at least 5% of the piles, which included remaining thickness readings taken on exposed sections of steel H-piles using an Ultrasonic Thickness Gauge (UT) on the web and flange. Underwater UT measurements were taken at mud line and low-water whenever possible. In some piles, these measurements could not be achieved due to obstructed access or severity of corrosion. High-water UT measurements were taken where concrete and timber jackets had spalled and fallen off the piles. Underwater visibility varied during the inspection but was approximately 2 to 5 ft. No dismantling or destructive testing of the structure was performed.

The dive team was a three-man team: Lead diver, dive team leader, and dive tender. The lead diver performed all UT measurements. The dive team leader provided additional in-depth underwater inspection services of the structure. The dive tender provided diver support through communication as well as topside support to ensure that other mariners and marine traffic were aware of the ongoing investigation.

Conditions were documented for all piles as defined in MOP 130 Element Level Damage Ratings (MOP 130 Tables 2-4 through 2-13) and Condition Assessment Ratings (MOP 130 Table 2-14). Refer to Tables 2 and 3 on the following pages which are developed from the previously referenced MOP 130 tables. Damage Assessment Ratings (see Table 2) were applied to timber, concrete, and steel components of the piles. Based on the assessment, a Condition Assessment Rating (Table 3) was established for each individual pile. It is important to note that while the steel pile is considered the structural element, the timber and concrete are intended as protective encasement. The condition of all elements was considered in establishing a Condition Rating due to the rapid increase in rate of steel deterioration that occurs once the timber and concrete elements have experienced advanced deterioration.

Table 2. Damage Assessment Ratings

Damage Rating	Damage Description
Not Inspected (NI)	Not inspected, inaccessible, or passed by.
No Defects (ND)	Sound surface material, light surface rust, no apparent loss of section or material.
Minor (MN)	Checks & splits, damage to protective coating, loss of thickness or section up to 15%.
Moderate (MD)	Loss of thickness or section 15% - 25%. More than 50% of surface affected by corrosion. Damage to hardware, loose bolts, noticeable hairline cracks, and splitting.
Major (MJ)	Loss of thickness or section 25% - 50%. Heavily corroded hardware, damaged coating or wrap, deteriorated edges, cracks up to ¼" in width, significant pitting.
Severe (SV)	Loss of thickness or section greater than 50%. Partial or complete breakage, structural buckling, loss of bearing, broken hardware, missing components.

*Damage Assessment Rating table shown is only meant to provide an understanding of the overall issues with the elements rated. For an in-depth understanding of the damage ratings refer to ASCE Manual 130, Chapter 2, Tables 2-4 through 2-13.

Table 3. Condition Assessment Ratings

Member Rating*	**Member Rating Description**
6 (A) Good	No visible damage or only minor damage is noted. Structural elements may show very minor deterioration, but no overstressing observed. No repairs required.
5 (B) Satisfactory	Limited minor to moderate defects or deterioration observed but no overstressing observed. No repairs are required
4 (C) Fair	All primary structural elements are sound but minor to moderate defects or deterioration observed. Localized areas of moderate to advanced deterioration may be present but do not significantly reduce the loading capacity of the structure. Repairs are recommended, but the priority of the recommended repairs are low.
3 (D) Poor	Advanced deterioration or overstressing 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 (E) Serious	Advanced deterioration, overstressing, or breakage may have significantly affected the load-bearing capacity of the 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 (F) Critical	Very advanced deterioration, overstressing, or breakage has resulted in localized failures(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.

*The letter assigned to the rating was added to the MOP 130 table to assist with field operations.

1.6 Hydrographic and Drone Survey

A hydrographic survey was completed in the vicinity of the pier over an area measuring approximately 13 acres that included the general area of the pier and extended approximately 150 ft beyond the pier in all directions. Survey data was collected on October 19, 2020 at high tide to allow for capturing survey over shallow and intertidal shoreline areas. The survey was performed with single beam sonar to map the sea floor on a 25-ft'by 100-ft sounding grid. The survey, in addition to mapping depths, was able to locate and map the toe of the rip rap slopes and any submerged debris including the rubble mound and remaining or intact areas of the submerged concrete slab beneath the northern pier.

A drone-based photogrammetric survey was also completed of the riprap shoreline, causeways, and immediate upland to tie in the hydrographic survey with upland topography. Drone survey was completed using a DJI Phantom Pro 4 drone and a FAA licensed drone pilot.

Control for hydrographic and drone survey was taken from three benchmarks set by CES, Inc. on October 16, 2020. The benchmarks are described in Table 4 below. Horizontal coordinates are based on NAD83 Maine State Plane East Zone, and elevations are referenced to MLLW = 0.0 ft vertical datum. All units are in feet.

Table 4. Survey Benchmarks

Benchmark	Location	Northing	Easting	Elevation
BM1	East end of north causeway	267203.28	1055876.59	+19.97
BM2	East end of south causeway	267059.98	1056122.69	+22.36
BM3	North pier deck near end of west vehicle bridge	267340.36	1056137.48	+21.85

A copy of the survey plan, which includes both hydrographic data in the vicinity of the pier and topography of the immediate, is provided in Appendix E.

1.7 Administrative Information

Primary contacts for this evaluation are listed below:

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2. Existing Conditions

Existing conditions are described within this section from the field inspection and data from historic plans for the facility. Inspection Tables with specific observations, condition ratings, ultrasonic thickness measurements, and electrical potential measurements are provided in Appendix C. Additional detailed inspection figures are provided in Appendix B. A photographic log from the inspection is provided in Appendix D. Appendices B, C, and D are referred to throughout this section. A schematic plan of the existing structure is provided in Fig. 2 below for reference. In the following section, Bent Lines and Pile designations are referred to by the gridlines depicted in Fig. 2.

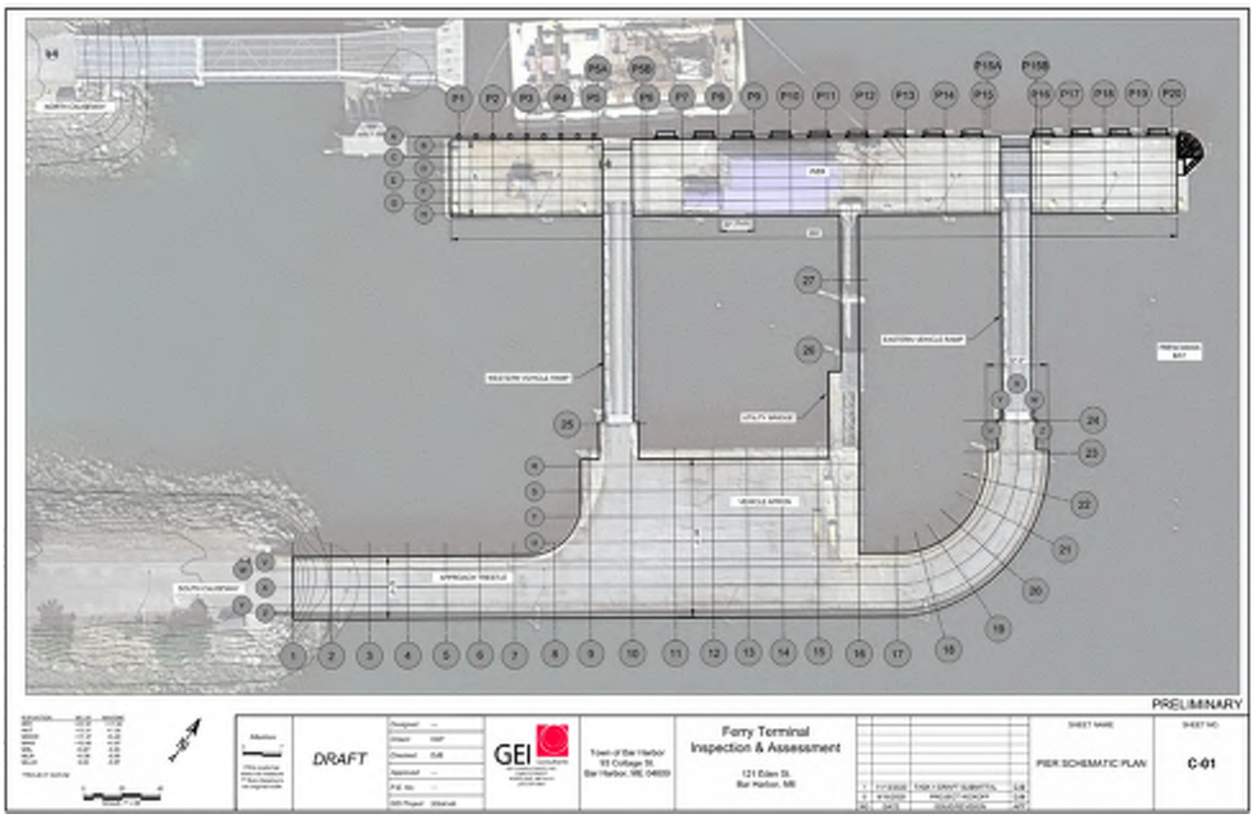

Fig. 2 – Schematic Plan of Pier

2.1 South Pier

The South Pier consists of an Approach Trestle that is approximately 32 ft 6 inches wide by 120 ft long, and a Vehicle Apron that includes a main section 86 ft 6 inches wide by 175 ft 8 inches long, and a curved access pier to the east vehicle ramp that is 32 ft 6 inches wide by 168 ft long.

The South Pier is supported on 124 total piles arranged in 25 pile bents with variable spacing of approximately 20 ft on center. Each bent consists of vertical and battered piles in arrangements of four (4) or seven (7) piles per bent. Piles typically consist of HP14x89 steel piles with concrete encasement from approximately 4 ft below the low water line up to the pile caps, and timber jacketing consisting of 3x8 planks held in place with steel through-rods. The out-to-out dimensions of the jacketed piles are approximately 24 inches square. Internal reinforcement in the pile encasement consists of two (2) steel reinforcing bars placed between the flanges on each side of the H-pile web, and a thin gage welded wire mesh surrounding the H-pile. Typical pile cross sections are shown in Fig. 3 below.

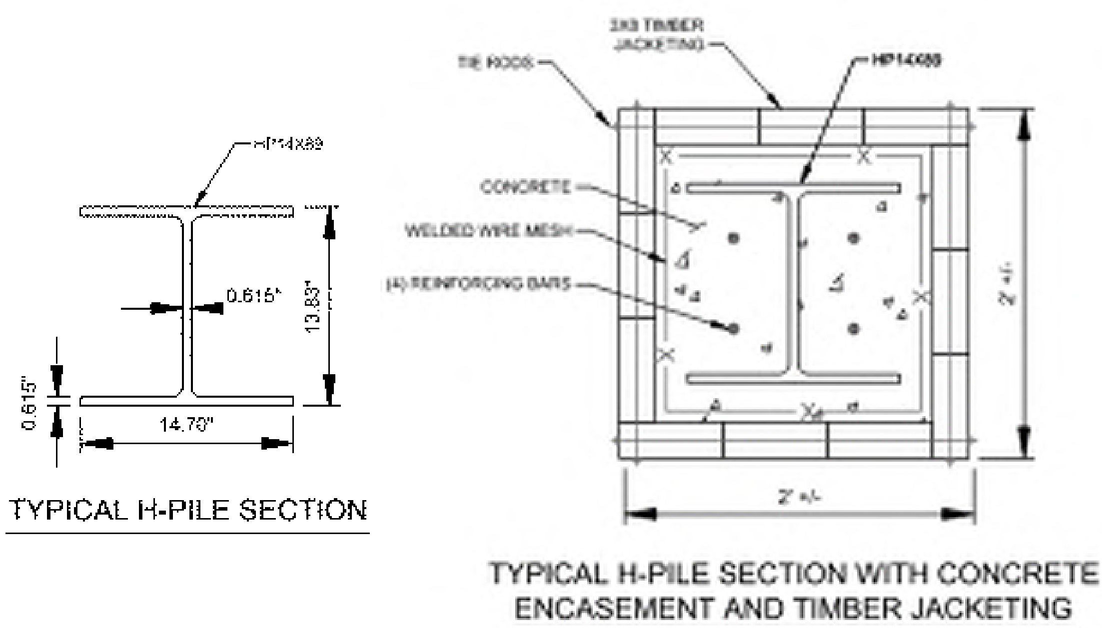

Fig. 3 – Typical Pile Configurations

The piles frame into concrete pile caps that are approximately 28 inches wide by 24 inches deep. The south pier deck is a 12 inch thick reinforced concrete slab with thickened edges to provide concrete curbs/sidewalks. An expansion joint is located above Bent 7. With the exception of locations where expansion joints were installed, the slab appears to have been cast integral with the pile caps. Design plans indicate that continuous reinforcement was provided across typical bents to provide flexural resistance above the caps. An asphalt wearing surface is present over the concrete deck. Steel guard rails and chain link fence are installed around the perimeter of pier deck on both sides. Deck drains are located at regular spacing along the face of the curb for drainage.

2.1.1 Topside Deck Conditions

The South Pier was paved throughout with asphalt. The asphalt pavement exhibited extensive cracking. Reflective cracking was present above each pier bent oriented parallel to the bent lines and on regular intervals located perpendicular to the bent lines. Vegetation was growing through the cracks in many locations (Photo 11). The topside of the concrete deck was not visually inspected due to the presence of asphalt wearing surface. Deterioration and spalling of the concrete curb was present in several locations, including:

- 5-ft spall near the eastern vehicle ramp.
- 2-ft spall above bent 16 on the southern side of the pier.
- 2-ft spall above bent 14 on the northern side of the pier.

Auditory inspection of the deck was undertaken by dragging a length of chain over the deck and listening to sound profiles to identify areas of delamination or separation. The asphalt wearing surface prevented from chain dragging directly on the deck. As a result, no notable observations were made from the auditory inspection that would identify specific areas of delamination. There is potential that areas of delamination exist that were not able to be identified using this inspection method.

Steel guardrails were intact with moderate corrosion. Chain-link fencing was generally in poor condition. On the eastern edge of the deck above Bent 16, the fencing was hanging off of the structure with vertical posts failed at the base (Photo 14). On the southern edge of the pier near Bent 1, a section of fencing was broken and had fallen over (Photo 13). Both of those conditions represent a fall hazard to users of the facility. A set of steel framed stairs was present on the north side of the pier adjacent Bent 1 which had broken free from the pier and was sitting loosely on the riprap revetment slope. The stairs are also a potential hazard.

There was a large sinkhole at the north end of the south causeway directly behind Bent 1. The sinkhole was approximately 3 ft by 4 ft wide at the surface and 4 ft deep (Photo 15). The sinkhole extends along the back of Bent 1 toward the south an unknown additional distance beneath the paved approach causeway. A clear opening to the front of Bent 1 was visible within the sinkhole which will create a path for further erosion. This poses a potential hazard to vehicles and pier users and should be barricaded to prevent access.

2.1.2 Underside Deck Conditions

Hairline cracking, efflorescence, minor spalling, and honeycombing of the concrete were typical of the underside conditions of the South Pier (Photo 25). More severe spalling and section loss was found in several locations on the underside of the deck and pile caps and is listed below:

- A 15 ft by 3 ft by 1 ft spall on Bent 2 (Photo 26).
- A 25 ft by 2 ft by 1 ft deep spall on the north side of the approach trestle from Bents 1 to 3 (Photo 27).
- A 2 ft by 2 ft spall on the northern edge of the structure between Bents 11 and 12.
- A 3 ft by 3 ft spall on the underside of the western vehicle ramp abutment.
- A 4 ft by 1 ft spall and a 3 ft by 1 ft spall on the northern side of the structure near Bent 14.
- A 2 ft 6 inch by 2 ft 6 inch spall just north of Pile 14-V.
- A 1 ft 6 inch by 1 ft 6 inch spall between Bents 15 and 16 just north of pile line W.
- A 2 ft 6 inch ft by 1 ft spall on the southern edge of the structure between Bents 16 and 17.
- A 7 ft by 1 ft spall on the northern edge of the structure between Bents 16 and 17.
- A 2 ft by 5 ft spall between Piles 22-Y and 23-Y.
- There was a total of approximately 143 square ft of spalling on the underside of the deck at the time of the inspection. General locations of these areas are shown in on Plan Sheet C-03 in Appendix B.

2.1.3 Steel Piles

The steel H-piles supporting the South Pier were in varying levels of degradation.

In the tidal range, timber jacketing had widespread rot and section loss varying from moderate to severe (Photo 37). In some locations, timber jacketing was partially broken/missing, had become detached and slid down the pile or was entirely missing (Photo 33).

In the areas where the timber jackets have deteriorated, concrete has typically spalled from outside the flanges of the steel H-piles and remains in some percentage of its original shape between the flanges (Photo 36). In several locations concrete within the flanges has deteriorated, exposing the internal reinforcement or has been lost entirely (Photo 34).

The piles with exposed steel had major pitting and moderate to severe corrosion throughout the exposed height. The area of highest corrosion was near the low water line which is typical of steel structures in marine environments. Historic records also indicate that many of the piles were field spliced near the low water elevation, which introduces an additional cause for accelerated corrosion. The majority of piles had various degrees of knife edging throughout the intertidal area with many areas of severe flange bites down to the web as well as many piles had holes in the web ranging from 4 inches to 10 inches in maximum dimension.

Remaining steel thickness readings were taken throughout on the webs and flanges of steel H-piles at mudline, low water, and high water where it was exposed. The average steel thickness reading was 0.422 inches, with a range of 0.275 to 0.550 inches. The full original thickness of the flange and web of the HP14x89 H-pile section is 0.615 inches. Using the recorded UT measurements, section loss ranges from 11% to 55% of original section dimensions with an average section loss of 33%. It is noted, however, that given the moderate to severe pitting of the steel surface some UT readings recorded higher remaining thickness than was observed in specific nearby areas with more critical deterioration (and in many locations 100% section loss has occurred). A full summary of ultrasonic thickness readings is provided in Appendix C.

The steel H-piles, concrete encasement, and timber jacketing were assigned Damage Ratings as defined in MOP 130 (see Table 2), and each pile was assigned a Condition Rating as defined in MOP 130 (see Table 3) based on these findings. Specific field notes and damage and condition ratings for each pile are provided in the tables in Appendix C. An overall summary of pile condition ratings for the South Pier is provided in Table 5 below. Pile conditions are depicted on Plan Sheet C-02 in Appendix B.

Table 5. South Pier Pile Condition Ratings

ASCE MOP 130 Condition Rating	A *Good*	B *Satisfactory*	C *Fair*	D *Poor*	E *Severe*	F *Critical*	NI Not Inspected	Total
Quantity of Piles	0	0	49	36	23	8	8*	**124**
Percentage	0%	0%	40%	29%	19%	7%	7%*	**100%**

*4 piles in Bent 1 unable to be inspected due to being embedded beneath riprap embankment. 4 piles in Bent 2 were visually inspected above water, however, are categorized as "Not Inspected" due to inability to inspect steel H-pile which was fully embedded within beneath the riprap embankment.

2.2 North Pier

The North Pier consists of a rectangular fixed pier 40 ft wide by 380 ft long that provides the primary berth for vessels using the facility along the north face.

The North Pier is supported on 156 total piles arranged in 24 pile bents with typical spacing of 20 ft on center. Each bent consists of vertical and battered piles in arrangements of 3, 6, 8, or 10 piles per bent. Piles typically consist of HP14x89 steel piles with concrete encasement from approximately 4 ft below low water up to the pile caps, and timber jacketing consisting of 3x planks held in place with steel through-rods. The out-to-out dimensions of the jacketed piles are approximately 24 inches square. Internal reinforcement in the pile encasement consists of 2 steel reinforcing bars placed between the flanges on each side of the H-pile web, and a thin gage welded wire mesh surrounding the H-pile. Typical pile cross sections are shown in Fig. 3.

The piles frame into steel pile cap beams consisting of W21x73 wide flange sections. The tops of the piles and the wide flange cap beams are embedded into and are integral with the concrete deck. The deck is a reinforced concrete slab that varies in thickness from 4 ft at the outer edges to 4 ft 6 inches at the center. The deck is reinforced with #10 bars at 18" on center on the top and bottom faces in each direction with 2" clear to all reinforcing. Timber curbs are installed on the outer edges of the pier, and chain link fencing is provided along the south face.

At the East vehicle bridge, there is a section of the pier approximately 17 ft 6 inches wide where the concrete deck slab ends, and the pier deck consists of bar grate decking supported on steel framing consisting of W16x26 wide flange sections in the east-west, W16x40 wide flange sections in the north-south direction at each outer edge, and W8x14 wide flange sections in the north-south direction at midspan. At the west vehicle bridge, a similarly sized opening on the concrete slab existed historically, but has since been filled in with a concrete slab as part of facility modifications completed in the early 1980s.

Mooring bollards are installed along both the north and south faces of the pier that are used by vessels when the main berth is occupied. Fendering for the main berth is installed on the north face of the north pier and consists of steel pipe piles and steel and HDPE fender panels mounted with compressible cylinder and leg-type rubber fenders. There are four (4) floating fenders with chain-tire netting that ride up and down this face over the tidal cycle that are secured to the steel framing with chains. The fendering components are the maintenance responsibility of Bay Ferries and are outside of the scope of this inspection.

At the northeast corner of the pier there is a corner pile cluster or turning dolphin consisting of approximately 32 timber piles with timber and steel framing.

A dilapidated timber building is located on the pier that historically supported passenger loading. Various aluminum and steel framed platforms and attachments are also located on the pier deck associated with the building and former ferry access ramps and were not part of the inspection scope.

2.2.1 Topside Deck Conditions

The north pier was constructed of concrete throughout the structure with a concrete surface. Cracking and surface spalling were typical of the topside conditions (Photo 7). During the auditory inspection several areas were identified to have probable surficial delamination. The areas of delamination were primarily located near the west vehicle bridge, and along the north face of the pier in front of the passenger building.

Bollards and other mooring structures were present throughout the North Pier. All bollards were recently repainted and as a result had no visible corrosion (Photo 8). The pier had a timber curb that ran along all faces of the pier, except for several openings along the north face presumed to be areas where vessel access was necessary. There was moderate to severe deterioration and rotting of the timber curbs on the eastern and western ends of the pier and along the south face behind the building (Photos 8 and 9). A section of steel guardrail was present on the east end of the south face of the pier that was partially detached at its base and was hanging of the side of the pier with exposed fasteners (Photo 10).

The North Pier had an abandoned dilapidated building in the middle of the that was historically used for vessel loading and unloading. On the west end of the pier there was an abandoned aluminum structure that was cut and moved from its original location. The building and aluminum structure were not considered in scope for this inspection.

2.2.2 Underside Deck Conditions

Hairline cracking, efflorescence, minor spalling, and honeycombing of the concrete were typical of the underside conditions of the North Pier (Photo 20). More severe spalling and section loss was found in several locations on the underside of the deck and pile caps and is listed below:

- An 80 ft by 4 ft spall from the western edge of the pier extending to Bent P5 (Photos 21 and 23).
- A 10 ft by 2 ft spall on the southern edge of the pier near Bent P1.
- A 1 ft by 1 ft spall along Bent P4.
- A 10 ft by 2 ft spall along the southern edge of the pier between Bents P4 and P5.
- Spalling along the length of the perpendicular bent extending between Bent P5A and P5B near Gridline C.
- A 3 ft by 1 ft spall along Bent P5B.
- A 6 ft by 1 ft spall between Bents P7 and P8.
- A 2 ft 6 inch by 1 ft 6 inch spall between Bents P7 and P8.
- A 2 ft 6 inch by 1 ft spall between Bents P8 and P9.

- An 80 ft by 2 f 6 inch spall between Bents P8 and P12 (Photo 22).
- A 15 ft by 2 ft spall between Bents P10 and P11.
- A 1 ft by 1 ft between Bents P12 and P13.
- A 5 ft by 2 ft spall from near Bent P14.
- A 2 ft by 2 ft spall between Bents P13 and P14.
- There was a total of approximately 620 square ft of spalling on the underside of the deck at the time of inspection. General locations of these areas are shown in Plan Sheet C-03 on Appendix B.

2.2.3 Steel Piles

The steel H-piles supporting the North Pier were in varying levels of degradation. Conditions on the North Pier were generally worse than the South Pier.

In the tidal range, timber jacketing had widespread rot and section loss varying from moderate to severe (Photo 37). In some location's timber jacketing was partially broken/missing, had become detached and slid down the pile, or was entirely missing (Photo 33).

In the areas where the timber jackets have deteriorated, concrete has typically spalled from outside the flanges of the steel H-piles and remains in some percentage of its original shape between the flanges (Photo 36). In several locations concrete within the flanges has deteriorated, exposing the internal reinforcement, or has been lost entirely (Photo 34).

The piles with exposed steel had major pitting and moderate to severe corrosion throughout their exposed height. The highest area of corrosion was near the low water line which is typical of steel structures in marine environments. Historic records also indicate that many of the piles were field spliced near the low water elevation, which introduces an additional cause for accelerated corrosion. One pile was completely split along a splice near low water line (Photo 31). The majority of piles had various degrees of knife edging throughout the intertidal area with severe flange bites down to the web as well as many piles had holes in the web. There were 12 piles that had been recently repaired by bolting new steel channel sections to the existing flanges over a length of 6 ft near the low water line. This repair was completed in 2019 by Cianbro Corp. The repairs installed on these piles had minor corrosion.

Remaining steel thickness readings were taken throughout on the webs and flanges of steel H-piles at mudline, mean low water, and high water where it was exposed. The average steel thickness reading was 0.441 inches, with a range of 0.325 to 0.588 inches. The full original thickness of the flange and web of the HP14x89 H-pile section is 0.615 inches. Utilizing the UT measurements, section loss ranges from 4% to 47% of original section dimensions with an average section loss of 28%. It is noted, however, that given the moderate to severe pitting of the steel surface some UT readings recorded higher remaining thickness than was observed in specific nearby areas with more critical deterioration (and in many locations 100% section loss has occurred). A full summary of ultrasonic thickness readings is provided in Appendix C.

The steel H-piles, concrete encasement, and timber jacketing were assigned Damage Ratings as defined in MOP 130 (see Table 2), and each pile was assigned a Condition Rating as defined in MOP 130 (see Table 3) based on these findings. Specific field notes and damage and condition ratings for each pile are provided in the tables in Appendix C. An overall summary of pile condition ratings for the North Pier is provided in Table 6 below. Pile conditions are depicted on Plan Sheet C-02 in Appendix B.

Table 6. North Pier Pile Condition Ratings

ASCE MOP 130 Condition Rating	A *Good*	B *Satisfactory*	C *Fair*	D *Poor*	E *Severe*	F *Critical*	NI Not Inspected	Total
Quantity of Piles	0	0	38*	60	43	15	0	**156**
Percentage	0%	0%	24%	39%	28%	10%	0%	**100%**

*Note that piles rated in Fair condition include the 12 piles repaired in 2019 by Cianbro.

A submerged concrete slab and rubble mound was part of the original construction installed beneath the north pier. In many locations the slab had failed and collapsed. In areas where the concrete had not yet failed there was spalling and cracking exposing the rebar which had severe corrosion. The broken concrete had fallen against the rubble mound beneath the pier against the north side but did not extend into the vessel berth.

2.2.4 North Timber Pile Cluster

The timber pile cluster/fendering on the northeast corner of the pier had evidence of fire damage as it had been severely burned and the connections that attached it to the pier have failed (Photo 9). The piles were swinging free at the tops, with normal wave action, and were only held in place by a steel cable that wrapped around the outer face of the piles and had been fastened to the pier deck. The timber piles had minor to no section loss below the waterline, but condition was severe near the tops due to the fire damage. The unsecured conditions could result in the piles becoming dislodged and creating an overhead hazard or a hazard to navigation.

2.3 Vehicle Bridges & Pedestrian Bridge

There were two vehicle bridges at the facility, labeled East Vehicle Bridge and West Vehicle Bridge, spanning between the North Pier and South Pier. The bridges consist of steel truss structures with spans of approximately 14 ft wide by 112 ft long with timber and steel grate decking. No evidence of structural deficiency was noted during visual inspection of the vehicular bridges. Based on record documents, the two bridges were recoated in the mid-1990s. The coating on the topside and underside of both ramps was in good condition, with localized areas of debonding and bubbling of coating, and moderate corrosion of internal steel bleeding through the coating in multiple locations (Photo 24).

Two additional steel H-pile bents were present that previously supported the pedestrian bridge (Bents 26 and 27). Bent 26 consisted of 2 battered H-piles with timber and concrete encasement similar to the pier piles, a concrete cap, and diagonal steel bracing similar which was also encased in concrete and timber. Bent 27 consisted of 2 battered H-piles that were not encased and had epoxy paint present from low water up to the pile cap, with a concrete cap and two rows of horizontal bracing located along their height.

The steel H-piles on Bents 26 and 27 has moderate to severe corrosion with missing/ failed timber and concrete, flange knife edging, flange bites, and holes in webs typical. Condition ratings for these piles ranged from C to F.

The historic steel pedestrian bridge is no longer present. The two pile bents that previously supported the pedestrian bridge currently are supporting steel framing and utilities. At the time of the inspection, all utilities were shut off to the facility. There was a steel pipe spanning the Bents that was broken and hanging into the water at high tide (Photo 18).

2.4 Shoreline and Approach Causeways

The entire shoreline of the property has been stabilized with riprap extending from below low water up to the upland grades. Riprap slopes are estimated to be 2 horizontal to 1 vertical. Riprap generally consists of angular stones with maximum dimension varying from of 1 ft to 4 ft.

The south approach causeway was constructed in the 1950s as part of the original construction of the ferry terminal. The south causeway is approximately 350 ft long extending from the natural shoreline to Bent 1 of South Pier. The construction is a filled earth causeway with riprap revetment slopes and asphalt pavement at the surface. Chain link fencing is installed on both sides of the causeway. No signs of global settlement or instability failure were observed, however minor surficial erosion was widespread. The rip rap slope had erosion along its length on both the north and south sides of the causeway where riprap had mostly been displaced from the slope and the underlying gravel is exposed. No geotextile or filter fabric was observed. Erosion of the south causeway at the west end of the south side was the most severe. Additionally, a sinkhole was found in the south causeway directly behind Bent 1 as described in Section 2.1.1.

The north approach causeway was constructed in the early 1980s for the addition of a transfer bridge for bow/stern loaded RO-RO vessels. The north causeway is approximately 220 ft long extending from the historic shoreline to the abutment for the RO-RO transfer bridge. The construction is a filled earth causeway with riprap revetment slopes and asphalt pavement at the surface. The causeway pavement, guardrails and fencing were in good condition and appear to be more recent. The riprap was intact with no significant erosion or signs of global settlement or instability noted.

3. Evaluation and Assessment

A structural assessment was completed on the existing pier in order to evaluate remaining live load capacity, and identify areas of deficiency, repair requirements, and recommended operational restrictions. This evaluation included:

- Analysis of typical pier piles in the North and South pier under vertical dead and live loading conditions.
- Analysis of North and South Pier deck slabs and pile caps under vertical dead and live loading conditions.
- Analysis of the North Pier for lateral loading resulting from mooring and berthing loads from the CAT on the North Berth.

The evaluation was performed with reference to the following codes and standards:

- AISC 360-10 – Specification for Structural Steel Buildings, Allowable Stress Design (ASD) for steel elements.
- ACI 318-14 – Building Code Requirements for Structural Concrete, Load and Resistance Factor Design (LRFD) for concrete elements.
- PIANC MarCom WG 33: Guidelines for the Design of Fender Systems and UFC 4-152-01 – Design: Piers and Wharves for vessel mooring and berthing analysis.

At the level of assessment undertaken for this project, the structural analysis was limited in depth and complexity. The analysis consisted of two-dimensional finite element modeling of pier bents using STAAD.Pro software, and evaluation of individual elements (piles, pile caps, and deck) based on tributary area and first-principles analysis. Assumptions made by during the analysis are noted in respective sections of the report. While a more complex three-dimensional analysis could potentially yield somewhat less conservative results in consideration of load distribution and structure redundancy, this level of analysis is not believed to be warranted at this time given the current condition and lack of use of the structure. Further, the recommendations for structural repairs that are described in Section 4 would be expected to remain similar even if a more refined analysis was completed.

The following sections describe the evaluation and summarize relevant findings.

3.1 Material Properties

Material properties utilized for the structural evaluation and assessment were as indicated on the available as-built construction documentation. Where information was unavailable, properties were assumed based on materials in use in the era of construction, historical specification references, and other past engineering reports and assessments of the facility. The following material properties were assumed:

Concrete:

- Unit Weight: 150 pounds per cubic foot (pcf)
- Compressive Strength at 28 Days: 3,000 pounds per square inch (psi)
    - Reference: Plan No. 10 “Pier Sections and Details” by Fay, Spofford & Thorndike Engineers and dated May 1954.

Steel Reinforcement:

- Unit Weight: 490 pcf
- Tensile Yield Strength: 33 kips per square inch (ksi) assumed
    - Reference: CRSI Engineering Data Report Number 48 “Evaluation of Reinforcing Bars in Old Reinforced Concrete Structures.”
    - Note: In the 2008 assessment by East Point Engineering (Reference No. 73), a yield strength of 280 MPa (40 ksi) was assumed. Although Grade 40 (Intermediate) steel was available during the era of construction, Grade 33 (Structural) was more typical for use as concrete reinforcement. Therefore, GEI has assumed the lower value to apply.

Pile and Beam Steel:

- Unit Weight: 490 pcf
- Tensile Yield Strength: 33 ksi assumed
    - Source: American Institute of Steel Construction (AISC) Design Guide 15 – A Reference for Historic Shapes and Specifications.
    - Similar properties were also assumed by East Point Engineering in 2008 assessment (Reference No. 73).

3.2 Gravity Loading Analysis

The gravity loading analysis consisted of an evaluation of the pier piles, deck, and pile caps under vertical dead and live loads. Sections were analyzed in their nominal (original) conditions, and with reduced section properties representing the range of deteriorated states observed during the field inspection.

3.2.1 Pile Capacity Evaluation

The existing pier piles were observed during the field inspection to vary in Condition Rating from *C – Fair* in which minor deterioration exists but does not significantly compromise the structural capacity of the element, to *F – Critical* in which severe deterioration exists to the point of near-complete or complete loss of section or breakage that would result in little to no remaining capacity. To assess the remaining capacity of existing piles an evaluation was completed that considered the pile performance in a range of deteriorated states.

The 1956 design plans indicate that the pier piles are 14BP89 sections. The grade of steel was unable to be located on the historic plans. A yield strength of 33 ksi was assumed which would have been typical of similar construction of the era based on AISC DG 15 and the 5th Edition AISC Steel Construction Manual (published in 1946). This is also consistent with the assumptions made by East Point Engineering in their 2008 analysis of the structure.

The compressive capacity was based on Allowable Stress Design (ASD) and evaluated in accordance with the Effective Length Method presented in AISC 360-10. Piles were assumed to be pinned at the pile cap and fixed at the base at a depth 10 ft below the mudline. The unsupported length of each pile varies by location within the pier. Nominal section properties were based on HP14x89 sections as summarized in Table 7, which closely align with the historic 14BP89 section properties.

Table 7. HP14X89 Nominal Section Properties

					Strong Axis				Weak Axis			
Area	Depth	Web Thick.	Flange Width	Flange Thick.	Moment of Inertia	Section Modulus	Radius of Gyration	Plastic Section Modulus	Moment of Inertia	Section Modulus	Radius of Gyration	Plastic Section Modulus
in^2	in	in	in	in	in^4	in^3	in	in^3	in^4	in^3	in	in^3
26.1	13.8	0.615	14.7	0.615	904	131	5.88	146	326	44.3	3.53	67.7

Section properties were calculated for pile sections ranging from 100% remaining thickness (no deterioration) to 30% of remaining thickness assuming uniform section loss over all faces (both sides of both flanges and both sides of web). These properties were used to determine the capacity of piles to carry axial compression loading over the range of states of deterioration and the range of installed lengths corresponding to the mudline depth measured in the field. The resulting distribution of pile capacities is shown in Fig. 4.

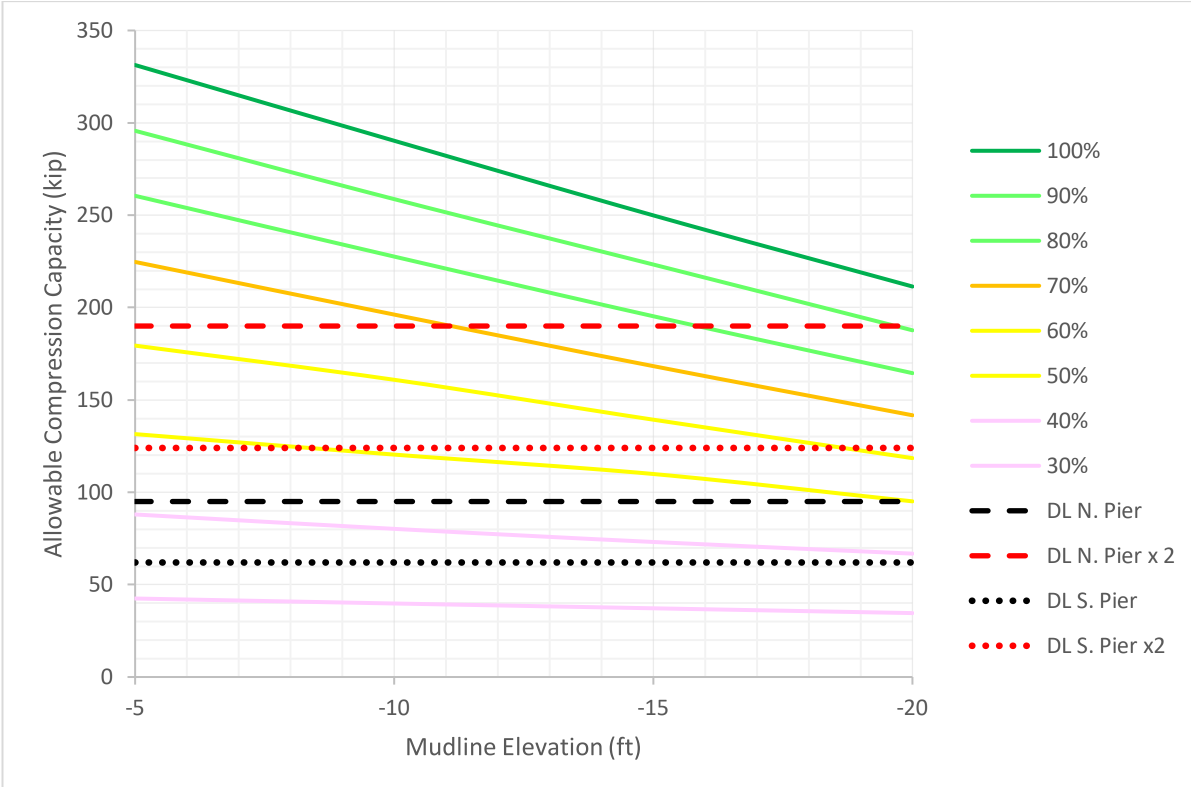

Fig. 4 – Allowable Pile Compression Capacity vs. Mudline Elevation for Typical Pile in Interior Pile Bent

Piles which had severe defects such as knifing, flange bites, hour glassing, or holes will have significantly lower remaining capacities than those presented in Fig. 4. Due to a more substantial reduction in cross-sectional area, inertial properties, and geometric irregularities within the cross section local instability failures may develop. Piles with these forms of deterioration were designated "Limited-Capacity Piles" (LC-Piles). For the purpose of the structural analysis, LC-Piles were considered ineffective and assumed to provide no support to the structure. The designation of an LC-Pile was applied to all piles assigned a condition rating of "E" or "F" during the field inspection. In total, this includes 89 piles (58 in the North Pier, and 31 in the South Pier), or 32% of all of the existing piles beneath the structure. In locations of LC-Piles, the nearest adjacent piles were assumed to support an increased portion of the applied dead and live load based on the resulting increased tributary areas.

Horizontal lines are plotted in Fig. 4 for the calculated pile Dead Load of a representative interior pile bent with all piles intact, based on typical bent configurations as shown in Sheets S-03 and S-07 for the North and South Piers, respectively, provided in Appendix B. A secondary line plots twice the pile Dead Load where an LC-Pile would result in an adjacent pile carrying twice the tributary area. In cases where multiple adjacent piles were designated

LC-Piles, the increased loading to adjacent piles is further increased in a similar manner. Each of these dead loads will vary by pile location and respective tributary area.

Key findings of the pile loading analysis are summarized in Table 8 below.

Table 8. Summary of Pile Loading Analysis Findings

Location/Condition	Minimum Remaining Thickness Required to Provide Adequate Capacity to Carry Structure Dead Load (% of Nominal Thickness)
North Pier	
All Adjacent Piles Intact	40-50%
(1) Adjacent LC-Pile	60-90%
(2) Adjacent LC-Piles	90-100%
More than (2) Adjacent LC-Piles	Inadequate regardless of condition
South Pier	
All Adjacent Piles Intact	35-40%
(1) Adjacent LC-Pile	50-60%
(2) Adjacent LC-Piles	70-85%
More than (2) Adjacent LC-Piles	85% to inadequate regardless of condition

The analysis described above considers the capacity of individual piles only. To further assess the implications of the pile analysis on remaining capacity of the structure globally, a live load analysis was undertaken on the pier deck to assess the remaining load capacity based on pile conditions. The analysis considered a uniform distributed live load over the pile tributary area, the condition of piles, and the presence of adjacent LC-Piles.

The original design live loading for the North and South Piers was unable to be identified from historic documents, however based on record plans, the original ramps between the piers were indicated to have been designed for H20-S16 and H15-S12 truck loading as well as passenger automobiles or light trucks spaced at 15 ft on center (Plan No. 8 "Steel Framing for Adjustable Car Ramp"). The applicable truck load diagram is depicted in Fig. 5 based on the 1944 AASHTO Standard Specifications for Highway Bridges. Given the need for access to the two vehicle ramps from the piers, it is assumed that at a minimum, the pier would have been designed for similar vehicular truck loading.

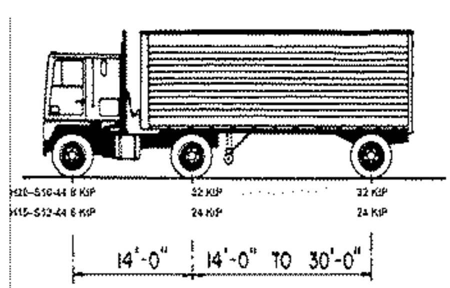

Fig. 5 – AASHTO H20-S16 and H15-S12 Load Diagram

Uniform live load requirements that may be considered for structures of similar use are summarized in Table 9 below from a variety of references. As an additional point of reference, the 2008 evaluation by East Point Engineering assumed a deck live load of 250 psf. Based on the known history of use, GEI assumes that the pier would have been originally designed for a minimum distributed live load of 250 psf.

Table 9. Summary of Live Load Specifications from Various Codes

Reference	Use/Activity	Minimum Live Load
ASCE MOP 50 – Planning & Design of Small Craft Harbors	Pedestrian • Restricted access • Unrestricted access Tractor-Trailer/Fire Truck	 50 psf 100 psf 250 psf
International Building Code (2015 Ed.)	Pedestrian Areas subject to trucking	100 psf 250 psf 8,000 lb concentrated
Unified Facilities Criteria 4-152-07; Design: Small Craft Berthing Facilities	Pedestrian • Restricted access • Unrestricted access Vehicular	 50 psf 100 psf Per UFC 4-152-01
Unified Facilities Criteria 4-152-01; Design: Piers and Wharves	Vehicular Fueling Berthing General Cargo	Per AASHTO 300 psf 600 psf 750 psf

For the North Pier and in areas where there are no LC-Piles, the piles are satisfactory for the Dead Load of the structure and have residual capacity for live load that varies depending on their condition. The estimated allowable live load based on pile condition/pile capacity for these areas is approximately 200 psf. However, in areas where adjacent LC-Piles are present, the pile load due to Dead Load can be double or more the nominal pile load and exceeds the allowable pile capacity. In cases where the dead load is found to exceed the allowable pile capacity, the allowable live load is 0 psf. Based on this analysis, it was found that a total of 66 piles beneath the North Pier have inadequate capacity to support any additional live load. The locations of these piles are depicted in Fig. 6 and on Sheet C-04 provided in Appendix B. Given the significant number of LC-Piles in the North Pier, the 'no remaining live load capacity' condition dictates the capacity for approximately 90% of the North Pier deck area.

For the South Pier and in areas where there are no LC-Piles, the piles are satisfactory for the Dead Load of the structure and have residual capacity for live load that varies depending on their condition. The estimated allowable live load based on current pile condition/ pile capacity for these areas is 100 psf. However, in areas where adjacent LC-Piles are present, the pile load due to Dead Load can be double or more the nominal pile load. The allowable live load in these areas is in the range of 0 psf to 50 psf. Based on this analysis, it was found that a total of 31 piles beneath the South Pier have inadequate capacity to support any additional live load. The locations of these piles are depicted in Fig. 6 and on Sheet C-04 provided in Appendix B. The 'no remaining live load capacity' condition dictates the capacity for approximately 40% of the South Pier deck area.

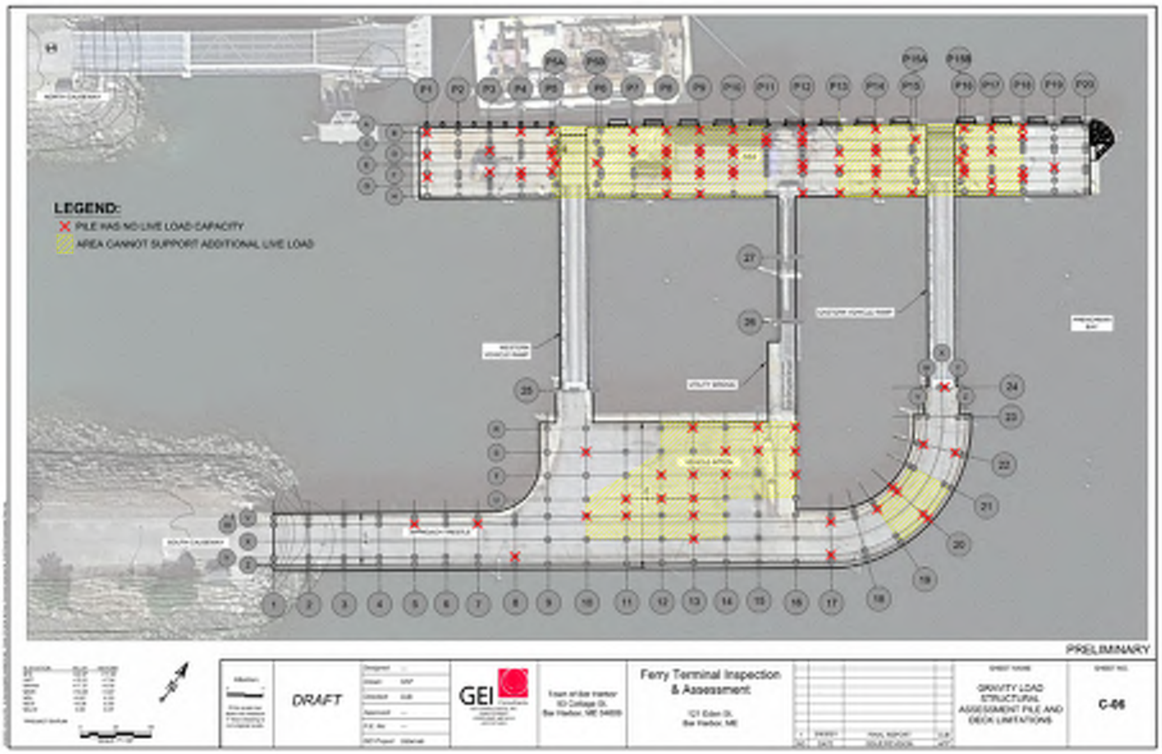

Fig. 6 – Locations of LC-Piles and No Remaining Live Load Capacity on Pier Deck

It should be noted that the analysis for remaining live load capacity includes code minimum factors of safety. Some LC-piles may retain a small amount of live load capacity, but are inadequate to satisfy minimum factors of safety and are therefore assigned a remaining live load capacity of zero.

The deteriorated state of the piles presents significant limitations to the allowable live load capacity of the pier deck. The implications of these issues will be discussed further in the following sections.

3.2.2 Reinforced Concrete Deck

The reinforced concrete deck was generally observed to be in fair condition, with localized areas of more severe deterioration including cracking, spalling, and exposed reinforcement. In areas where reinforcement was exposed, the reinforcement had surface rust and minor section loss. The conditions have a moderate impact on global design strength, and will have an impact on the future durability of the structure. Further neglect to repair the structure will result in accelerated deterioration and reduction to the deck capacity.

The concrete deck of the North Pier varies in thickness from 4 to 4.5 ft thick and was reinforced with #10 bars at 18 inches on center (OC) top and bottom and placed in both directions. Two inches of protective concrete cover was specified on all faces of the slab. A typical transverse cross section of the deck is shown as Fig. 7.

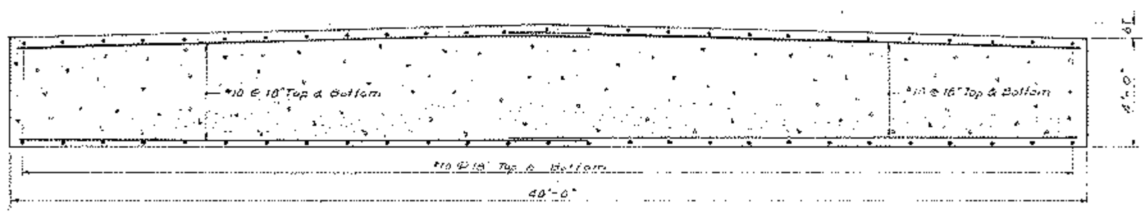

Fig. 7 – North Pier Typical Transverse Deck Section

The pier deck was analyzed as a continuous one-way slab with supports at the pile-supported bents and a span length of 20 ft 9 inches (the maximum bent spacing in the North Pier). The deck was analyzed based on its original conditions and full pile support and was calculated to have a live load capacity upwards of 750 psf. However, the amount of reinforcement is below the code required minimum for flexural reinforcement based on modern codes, which suggests the original design loading may have been less, and the deck geometry and reinforcement layout may have been selected for crack control and/or to accommodate configurations of structural attachments and embedded items.

The concrete deck of the South Pier is not as well documented as the North Pier. Our understanding is based on limited construction documentation for the Approach Trestle. No construction documentation for the Vehicle Apron has been located. The slab construction is assumed to be similar to the Approach Trestle. The deck is a 12-inch-thick concrete slab with two inches of bituminous concrete overlying the slab. The steel reinforcement varies considerably and is as shown in Fig. 8. One- and one-half inches of protective concrete cover was specified. Based on the presented cross section, a representative reinforced concrete deck section was analyzed based on the following:

- Crack Control Top Reinforcement: #4 @ 12 inch OC
- Crack Control Bottom Reinforcement: #5 @ 12 inch OC
- Flexural Bottom Reinforcement (between bents): #7 @ 8 inch OC
- Flexural Top Reinforcement (at bent): #7 @ 8” OC

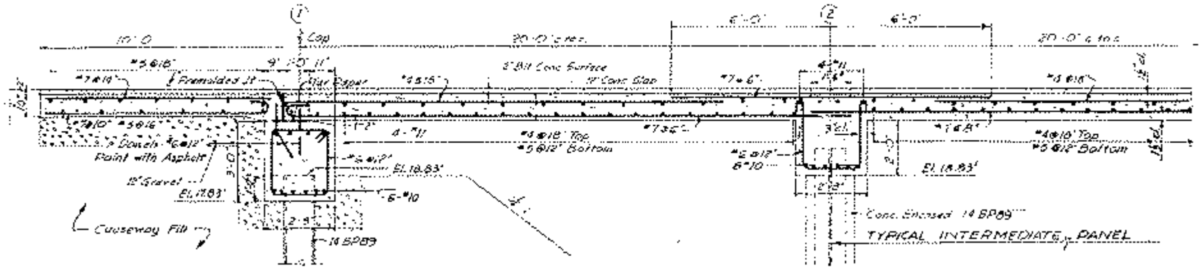

Fig. 8 – South Pier Approach Trestle Longitudinal Deck Section

The pier deck was analyzed as a continuous one-way slab with supports at the pile-supported bents. A typical span length of 20 ft was used. The deck was analyzed based on its original conditions and full pile support and was calculated to have a live load capacity of approximately 250 psf. This is consistent with uniform live loads in areas subject to traffic loading, as described previously and shown in Table 9.

The type and extent of deterioration currently present in the deck would generally result in concerns pertaining to the durability and longevity of the structure. The sporadic and varied nature of deterioration would also result in a reduction in capacity which varies by location, type, and extent of deterioration. During the field inspection, the pier deck was generally observed to have conditions classified as "Moderate" based on ASCE 130 damage ratings. Based on these conditions, and in lieu of a more detailed analysis, it is recommended that a capacity reduction of 30% be taken from the estimated original loading to account for current conditions. This would result in a maximum live load capacity in present conditions of 175 psf based on the deck capacity. This capacity would fail to meet the required live loading for vehicular access but would be adequate for pedestrian loading in locations where piles are intact and effectively supporting the pier. In locations of LC-piles, the deck capacity is further limited due to the piles.

The next phase of deck analysis considered the capacity of the deck to support increased spans due to the presence of LC-Piles. The loss of pile support significantly reduces the capacity of the pier due to increased stresses and potential instability that result, as well as potential for applied stresses not intended by the original design. As a result, the live load capacity for portions of the deck that do not retain effective pile support due to the presence of one or more LC-Piles is reduced to zero. Based on this analysis, the following areas of the pier deck are found to have no remaining live load capacity:

- North Pier between bents P5 and P11.
- North Pier between bents P13 and P18.
- South Pier over an area encompassed by the following grid points moving in a clockwise direction: 10X, 10U, 12S, 12R, 16R, 16U, 14U, 14X.

- South Pier between bents 19 and 21.

These areas are depicted in Fig. 6 and on Sheet C-04 provided in Appendix B.

3.2.3 Pile Cap Capacity

To transfer the load between the concrete deck and piles an intermediate structural beam is used. On the North Pier, the structural beam is a wide-flange W21x73 beam embedded within the concrete slab and resting atop of the piles. Cross-sectional properties of the cap beam are summarized in Table 10 below.

Table 10. W21X73 Cross-Sectional Properties

Area	Depth	Web Thick.	Flange Width	Flange Thick.	Strong Axis				Weak Axis			
					Moment of Inertia	Section Modulus	Radius of Gyration	Plastic Section Modulus	Moment of Inertia	Section Modulus	Radius of Gyration	Plastic Section Modulus
in^2	in	in	in	in	in^4	in^3	in	in^3	in^4	in^3	in	in^3
21.5	21.2	0.455	8.30	0.74	1600	151	8.64	172	70.6	17.0	1.81	26.6

The material properties assumed for the piles were also assumed for the wide-flange beams. While the condition of the pile caps was unable to be observed during the inspection, limited deterioration of these sections is expected due to the intact concrete encasement. Based on the assumption that no significant deterioration has occurred to the beams, an allowable live load of 250 psf was estimated.

On the South Pier, the pile caps are reinforced concrete and cast integrally with the concrete slab as shown in Fig. 8. The beams are 2 ft 8 inches wide and 3 ft 0 inches tall including the deck thickness. The beams have top and bottom reinforcing layers consisting of 4 - #11 and 6 - #10 bars respectively. Stirrups are #6 bars spaced at 12 inches on center. A uniform service live load of 735 psf was estimated based on the flexural capacity of the beam. This is nearly three times larger than the capacity of the deck, so it is assumed that the more conservative value would have applied and the design may have been developed considering conditions for crack control and geometric configurations.

3.2.3.1 Conclusions of Vertical Loading Analysis

A capacity summary of the structural elements discussed above is provided as Table 11. It is assumed that the original pier was likely designed for vehicular traffic loading equivalent to a uniform live load of 250 psf. Given the significant deficiencies of the piles which severely limit capacity of the pier deck, there are considerable areas with no remaining live load capacity. As a result, the current recommended load rating for the pier is 0 psf. In the current state, the pier should be closed to vehicular and pedestrian traffic to minimize potential risk to the public. Repairs to the piles could allow some of the capacity to be restored to the existing limited-capacity piles, which would in-turn increase the capacity of the deck, however limitations on vehicular and pedestrian loading may still be required depending on the extent of repairs undertaken.

Table 11. Gravity Load Capacity Summary

Element	Estimated Original Capacity (psf)	In Current Condition		
		Estimated Capacity (psf)	Adequate for Pedestrian Loading	Adequate for Vehicular Loading
Piles		0 - 400	No	No
No Corrosion	600 - 1,000*			
50 Year Design Life**	250 – 400*			
South Pier Deck				
Based on Deck Capacity	250	175	Yes	No
With Consideration for loss of support at LC-Piles		0 - 175	No	No
North Pier Deck				
Based on Deck Capacity	750	525	Yes	Yes
With Consideration for loss of support at LC-Piles		0	No	No
South Pier Concrete Pile Caps	735	500	Yes	Yes
North Pier Steel Pile Caps	250	250	Yes	Yes
South Pier Overall Load Rating	250	0	No	No
North Pier Overall Load Rating	250	0	No	No

*Uppermost bound – does not include lateral loads (wind, waves, mooring, berthing, etc.) in combination with vertical loads
** Assumes corrosion rate of 0.003 inch per year (ipy) in the splash zone and 0.0014 ipy in the immersed zone and concrete jackets are effective for 25 years (ASCE Manual 130)

3.3 Lateral Berthing and Mooring Analysis

A lateral load analysis of the North Pier was undertaken to investigate the feasibility to continue to support the short-term continued use by the CAT in the North Berth of the facility. This analysis consisted of:

- Berthing Energy Analysis for the CAT, and for the original design vessel, the MV Bluenose.
- Investigation of fender system capacity and load transfer mechanisms.
- Investigation of the mooring system capacity.
- Modeling of pier in STAAD.Pro to determine lateral loading limits.

3.3.1 Vessel Parameters

The berthing energy of The CAT and the original design vessel MV Bluenose were evaluated in accordance with PIANC MarCom WG 33: Guidelines for the Design of Fender Systems. The parameters used in the evaluation of each vessel are summarized in Table 12. Sources for these parameters for The CAT are from the Bay Ferries website, the U.S. Navy Military Sealift Command website, and online MarineTraffic database for the vessel, formerly known as the "Alakai" and the "HST-2." Photos published on the NavSource Naval History website aided in assessing windage areas for review of the mooring loads. Sources for parameters for the MV Bluenose were sourced from an unnumbered drawing titled "Dredging – Plan, Sections, Ship Section & Profile" and dated September 14, 1984 for the "Completion of Ferry Terminal Renovations" Project, and from the Wikipedia page for the MV Bluenose.

The calculated berthing energy for The CAT was used to assess the fender system and determine applied loading on the pier. The berthing energy of The CAT was also compared with that of the MV Bluenose to assess demand in comparison to probable original design conditions. The berthing energy of The CAT was found to be less than half that of the MV Bluenose, with an assumed approach velocity that is approximately double.

Table 12. Vessel Parameters for Berthing Analysis

Parameter	The CAT	MV Bluenose
Displacement	1,646 Ton	7,280 Ton
Length	349 ft	346 ft
Beam	78 ft	65 ft
Draft	12 ft	18 ft
Block Coefficient	0.16 (calibrated based on published displacement)	0.55 (assumed lower bound from PIANC)
Approach velocity perpendicular to pier	0.78 fps	0.41 fps
Approach Angle	15 deg	15 deg
Forward Velocity	3 fps (1.8 knot)	1.6 fps (1 knot)
Abnormal Impact Factor	2	2
Calculated Berthing Energy	30 kip-ft	69 kip-ft

3.3.2 North Berth Fender System

The configuration of the fender system varies along the North Berth Face. Between bents P6 and P20 the fender system is as shown in Fig. 9. This section of fender system consists of fender frames constructed of two vertical W24x104 piles spaced 10 ft on center with transverse struts connecting the piles. At the base, the piles are driven into the seabed. At the deck elevation, a TrellexMorse MV400x1000 fender element is installed between the pile and face of the concrete. The frames are installed at approximately 20 ft on center. At four locations, floating pneumatic fenders are also installed in front of the fender panels. The floating pneumatic fenders are approximately 5 ft in diameter by 10 ft long (1.5mx3m) with chain-tire net lattice encompassing the pneumatic fender. For the purpose of this analysis, the fender is assumed to be a Trelleborg 1500x3000 Pneumatic Fender inflated to 80 kPa, or similar.

Between bents P1 and P5 the fender system is as shown in Fig. 10. The fender system consists of an 18-inch diameter steel pipe pile that is driven into the seabed at the base and attached to the pier deck at the tops. At the deck elevation, 20-inch diameter cylindrical extruded fenders are installed between the pile and concrete stanchions. The fender piles are installed at 24 ft on center. The fender piles have UHMW panels fastened to the berthing face throughout the range of vessel contact elevations. For the purpose of this analysis, the fender is assumed to be a Trelleborg 500x250 Cylindrical Fender, or equivalent. A summary of the energy and reactions of each fender unit is provided in Table 13.

Fig. 9 – Fender System From Bent P6-P20

Fig. 10 – Fender System West of Bent P6

Table 13. Summary of Fender System Component Capacity

Fender Unit	Energy (kip-ft)	Reaction (kip)
TrellexMorse MV400x1000	20.0	34.0
Trelleborg Pneumatic* 1500x3000	157.8	171.0
Trelleborg Pneumatic Oper* 1500x3000	30.0	32.0
Trelleborg Cylindrical 500x250 – 380mm long	7.4	22.3

*The pneumatic fender has significantly greater energy absorption capacity than the calculated berthing energy from The CAT. A reduced value is used based on the published energy curves to determine the operational reaction assuming that the full berthing energy is carried by the Pneumatic Fender.

Under regular service conditions, the applied energy to the fender system resulting from berthing of the CAT will be distributed among the fender system components in proportion to their relative flexibility and based on the angle of approach, area of vessel contact, and hull stiffness. As a conservative approach to evaluating the applied loading on the pier resulting from berthing energy absorption, it is assumed that the limiting condition for maximum loading on the pier structure will not exceed the maximum reaction of the TrellexMorse fender units that attach the tops of the steel fender panels to the pier deck. Based on this assumption, the maximum fender reaction of 34 kips is assumed to be the maximum berthing force on the pier deck.

3.3.3 Mooring System and Mooring Loads

The mooring system consists of ten cast steel mooring bollards on the North Pier with six located along the north face and four along the south face. The base of the bollard is approximatle 2 ft 10 inch square and mounted to the pier deck with eight 1.25 inch diameter bolts. No capacity of the mooring system is provided on the construction documentation, and no marking were observed on the bollards that would identify the bollard capacity, however based on fixtures of similar style and bolting arrangements it is estimated that the capacity of the mooring bollards is approximetly 50 tons.

In lieu of performing an extensive vessel mooring analysis, such as Optimoor, a simplified mooring analysis was undertaken based on wind loading of the CAT when moored to the pier. An assumed mooring configuration was analyzed that consisted of a set of bow lines and a set of stern lines each moored to bollards located along a single bent line. The applied mooring line loads were applied to the nearest bent with no load distribution considered between bents. This is a conservative approach as the concrete deck slab will tend to distribute loading among adjacent bents, however the lack of continuity between Bents P5/6 and Bents P15/16 resulting from the locations of the former movable vehicle bridges means that a limited number of bents will contribute to lateral load resistance. Wind loading on the CAT was calculated based on UFC-4-152-01 - Design: Small Craft Berthing Facilities. A broad side windage area of approximately 22,700 square ft was estimated. A maximum operational wind speed of 30 miles per hour was assumed. This yielded a total mooring load of 91 kips which was assumed to be split evenly between the bow and stern moorings (40.5 kips each).

3.3.4 Structural Analysis and Assessment

To evaluate the lateral load capacity of the North Pier structure for loads resulting from vessel mooring and berthing, a 2-dimensional plane frame analysis of the North Pier was completed using STAAD.Pro, a commercial grade finite element analysis program. A single typical bent with a tributary width of 18 ft 10 inches was used which is representative of average bent spacing. The bent model is depicted in Fig. 11. The piles and pile cap were modeled as frame elements. Fixity was assumed at a pile penetration of 10 ft. No fixity was assumed at the pile top connection to the pile caps/concrete deck. Gravity loads included in the model include the self-weight of the piles, beam, and concrete slab. No vertical live loads were considered acting on the deck surface. Lateral loads, discussed and computed above, were input as concentrated loads at the deck elevation which could act in either direction (vessel loading toward or away from the pier). The resulting maximum pile loads for the lateral analysis are shown in Fig. 12. The resultant loading is within the allowable limit for piles in poor condition and with up to 40-50 percent section loss (60-50% remaining thickness).

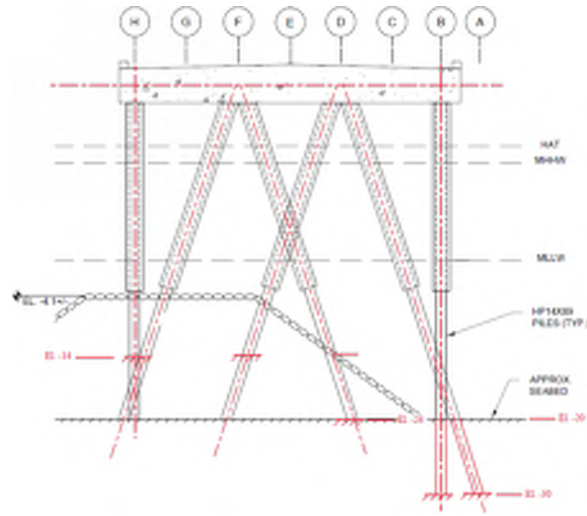

Fig. 11 – Typical Analyzed Bent

Pile conditions and dead load demands were reviewed in Bents P1/P2 and P19/20 to resist the lateral mooring loads. These bents correspond to the locations of the existing bollards that would be used for bow and stern lines from the CAT. On the west end, the combination of Bents P1 and P2 is found to have sufficient piles with adequate remaining capacity to resist the applied mooring load, although some piles fail to meet the required criteria of 50-60% remaining thickness. On the east end, the combination of Bents P19 and P20 is found to have sufficient piles with adequate remaining capacity to resist the applied mooring load, although some piles fail to meet the required criteria of 50-60% remaining thickness.

The location of the fender load can occur anywhere along the Berth Face and the vessel could impact a severely deteriorated area such as between Bents P8 and P10. However, the concrete slab acts as a diaphragm and has considerable capacity to redistribute and share loading between bents which meet the criterion established for the mooring analysis. Given the redundancy of the structure, and the ability of the deck to redistribute the load, the condition of the structure is likely sufficient resist the applied lateral mooring and berthing loads provided that the maximum used conditions remain within the operational conditions considered. However, to minimize potential for overstressing of more deteriorated bents it is recommended that berthing be undertaken so as to limit concentrated impact to portions of the North Pier between Bents P8 and P11, where the most severe deterioration exists. Additionally, a reduced berth velocity is recommended to provide additional factor of safety on applied berthing energy.

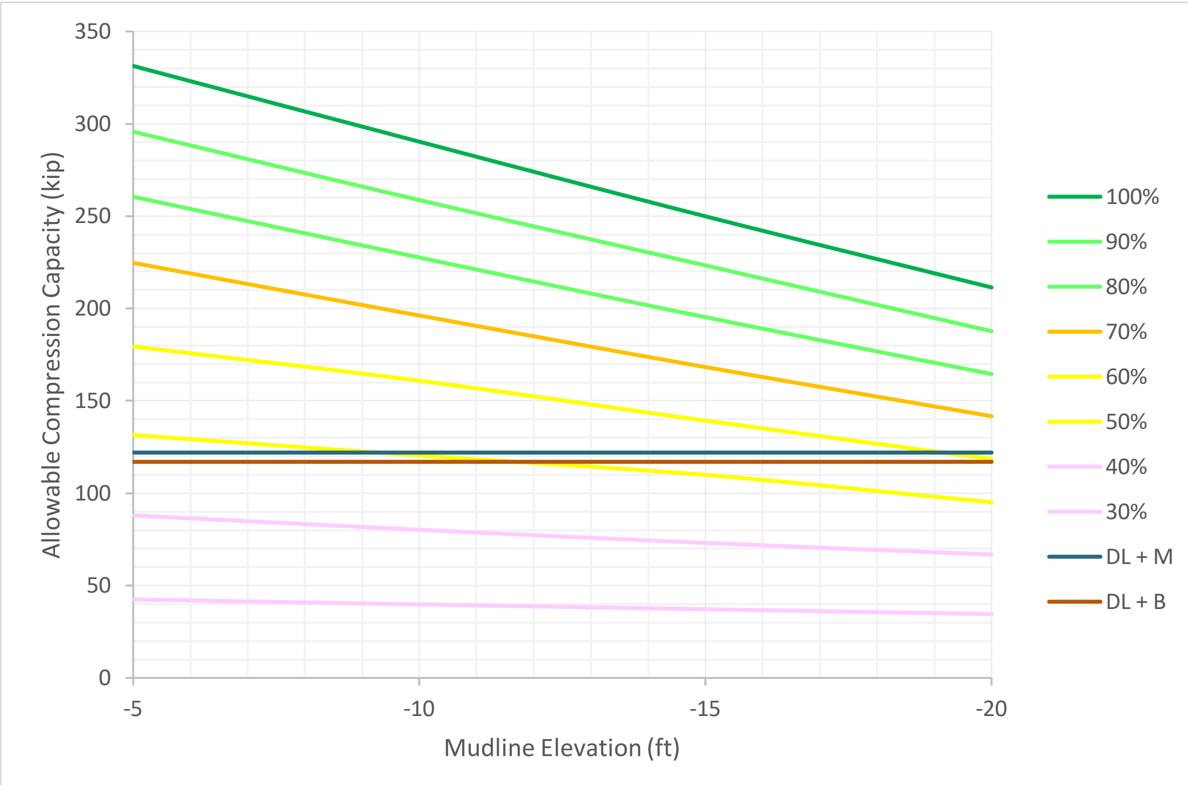

Fig. 12 – Axial Pile Capacity and Berthing/Mooring Loads

3.3.5 Berth Limitations

Based on discussions with the Harbormaster GEI understands that when the CAT uses this facility, its movements are in a slow and controlled manner and if weather conditions are poor, trips typically will not occur. Therefore, it is understood that limitations on berth approach velocity and maximum wind speed are likely to be acceptable to Bay Ferries for their continued short-term use. The following limitations are recommended for the North Berth and should be implemented to allow for continued use of the facility by the CAT.

1. Maximum Berthing Velocity: 1 knot
2. Maximum Berthing Approach Angle: 15 degrees
3. Maximum wind speed while at berth: 30 mph

4. To the maximum extent possible, avoidance of concentrated berthing impact on Bents P8 to P11.

5. No public access live load on the ferry terminal.

These recommended operational restrictions should be reviewed with Bay Ferries to confirm that they are not overly prohibitive to Ferry operations. The various assumptions made regarding characteristics of the CAT should also be confirmed by Bay Ferries to validate the analysis.

4. Recommendations

As documented by the inspection and evaluation, the existing pier is in a deteriorated state which results in reduced structure capacity and required limitations on operational loading. In order to maintain safe facility operations, recommendations are made within the following section for:

- Short-term operational recommendations (immediate).
- Short-term repairs (next two to three years if pier is to remain in service).
- Long-term repairs (next three to five years if pier is to remain in service).
- Long-term reuse.

4.1 Short-Term Operational Recommendations

Recommended Timeframe: Immediate

The current deteriorated conditions result in reductions to pier capacity. The following practices are recommended immediately to support safety to the public and users of the facility in the short-term:

1. Close the facility to public access by pedestrians or vehicle traffic. GEI understands that this step was completed in late fall 2020.
2. Review operational limitations with Bay Ferries to maintain use within the following maximum recommended parameters:
    a. Maximum berthing velocity of the CAT: 1 knot
    b. Maximum berthing approach angle of the CAT (hull to berth face): 15 degrees
    c. Maximum wind speed while at berth: 30 mph
    d. To the maximum extent possible, avoidance of concentrated berthing impact on Bents P8 to P11.

In the short-term, GEI believes that these limitations will allow the Town to leave the pier in service for the remainder of Bay Ferries current lease term (until 2024). However, as the structure continues to deteriorate it will be important to monitor conditions and revisit operations as conditions deteriorate further. Localized repairs or further operational restrictions may be warranted if conditions have worsened.

The North Pier is currently overall in poor condition with various areas in serious and borderline critical condition. ASCE MOP130 recommends a maximum interval between routine inspections of one to three years for structures in serious to poor condition. Given the intent to continue using the North Pier for the CAT, we recommend routine inspections be performed every two years for the North Pier going forward. The Town should plan for the next inspection to occur in 2022. Because the most severe deterioration is related to the piles, it will be important that an underwater inspection be completed at this time. At this time, we are not recommending any further routine inspections of the South Pier as we understand the facility is closed to the public, however additional inspection would be warranted as part of a repair program design if the facility is to be reopened for a future use.

4.2 Short-Term Repairs

Recommended Timeframe: Next two to three years if pier is to remain in use.

If the town decides to reopen the facility to the public, the following short-term actions and repairs below are recommended.

- Perform a Structural Repair Inspection one year from anticipated start of construction. The Structural Repair Inspection is significantly more detailed than a Routine Inspection and is intended to develop repair bid documents. The interval between the inspection and construction should be minimized to more accurately reflect the repair extents.

- Based on the results of the Structural Repair Inspection, repair all piles rated E and F with full-height concrete jackets and installing steel inserts at locations with major section loss. Repair any locations of major concrete deterioration, as identified in the inspection, which severely limit the capacity of the deck. These repair approaches are elaborated in Section 4.4. Based on the results of the routine inspection, we believe that the majority of concrete repairs could be deferred until the Long-Term Repairs are implemented.

- Repair public safety and functional issues, including, but not necessarily limited to: repair of sinkhole at approach to south pier, repair of failed fencing and guardrails, restoration of public safety elements (life rings, fire extinguishers, etc.), removal of detached/hazardous components (stairs adjacent south pier, hanging utilities, hanging guardrail, damaged timber piles on north pier, etc.). A full scope of repairs will need to be developed considering the intended future use of the site, and may also involve further upgrades.

4.3 Long-Term Repairs

Recommended Timeframe: Prior to facility reopening/reuse.

- Perform a Structural Repair Inspection one year from anticipated start of construction. This inspection could be performed in conjunction with construction and baseline inspections of the Short-Term Repairs.

- Based on the results of the Structural Repair Inspection, repair the remainder of the piles with full-height concrete jackets and installing steel inserts at locations with major section loss. Many of these repairs should be limited to the concrete jacket unless the interval between Short-Term and Long-Term Repairs lags considerably. These repair approaches are elaborated in Section 4.4.

- Repair the remainder of the concrete deck sections to restore lost concrete and reinforcement, and reduce potential for accelerated future deterioration.

- Resolve any remaining public safety, utility, or functional issues.

4.4 Considerations for Repair or Replacement

As structures approach the late stages of their service life considerations should be made to what their future use may be. Changing future uses have the potential to alter structure loading and operational requirements and should be considered in evaluating rehabilitation or replacement options.

Considerations for future structure improvements generally fall into three categories based on level of deterioration and future operational requirements of the facility. These categories are as follows:

- Preservation: No structural upgrades are required; Restoration is made to protective measures only.

- Rehabilitation: The structure requires structural reinforcement due to deterioration or operational requirements in order to restore lost capacity.

- Replacement: The structure is deteriorated beyond the point of repair or rehabilitation is not cost effective to meet the future operational requirements.

    - Replacement in-kind: a new structure of the same footprint and dimensions is provided to provide similar function to the deteriorated structure.

- Replacement/redesign: A new structure is designed that meets operational parameters related to a new/different use of the site than the existing structure was designed for.

In the case of the ferry terminal piers, it is the opinion of GEI that the structure has surpassed the point at which preservation-type repairs are possible. A potential program of rehabilitation-type repairs is described in the following section. This rehabilitation would be intended to restore the pier to maintain use similar to the original design conditions. If future uses are to change, then consideration should be given to which portions of the structure remain compatible with future use, and applicable design and loading requirements.

4.4.1 Pile Repair Concepts

Pile repairs fall into two categories: non-structural which are preservation type repairs and structural which are rehabilitative type repairs. In both instances, they can provide an additional 25 years of protection against the marine environment. A non-structural type repair is shown as Fig. 13 and consists of a stay-in-place fiberglass form centered on the pile and filled with cast-in-place concrete. The concrete is lightly reinforced with temperature and shrinkage steel to limit cracking of the concrete which would allow intrusion of chlorine ions. The concrete jacket is generally sacrificial and not relied upon for strength, however, could be designed for structural contribution as well if necessary. Final sizing of the concrete jacket and reinforcement would be refined during detailed design. To install the jackets, a contractor would need to remove the existing jacket, including any unsound concrete, and clean off marine growth from all faces of the pile for its full height. The contractor would then place the rebar and fiberglass shell around the pile and inject concrete into the annular space. Much of this work would be completed in the tidal and permanent immersion zones and would require divers to perform the work. Because of this, the repair is labor intensive and time consuming which is often reflected in the construction cost. A preservation repair is applicable to piles which have no substantial defect other than general corrosion within acceptable limits.

Piles which have substantial defects and considerable section loss can be rehabilitated using a structural type repair as shown in Fig. 14. The repair includes the same elements as the non-structural jacket but has an added steel insert which consists of steel channels located on either side of the web and attached with bolts. The length and location of the steel insert must be determined on a case-by-case basis to encompass the areas of most severe deterioration. At minimum, it is anticipated that the steel insert would be approximately 5 ft long, but a longer length may be required based on extent of deterioration.

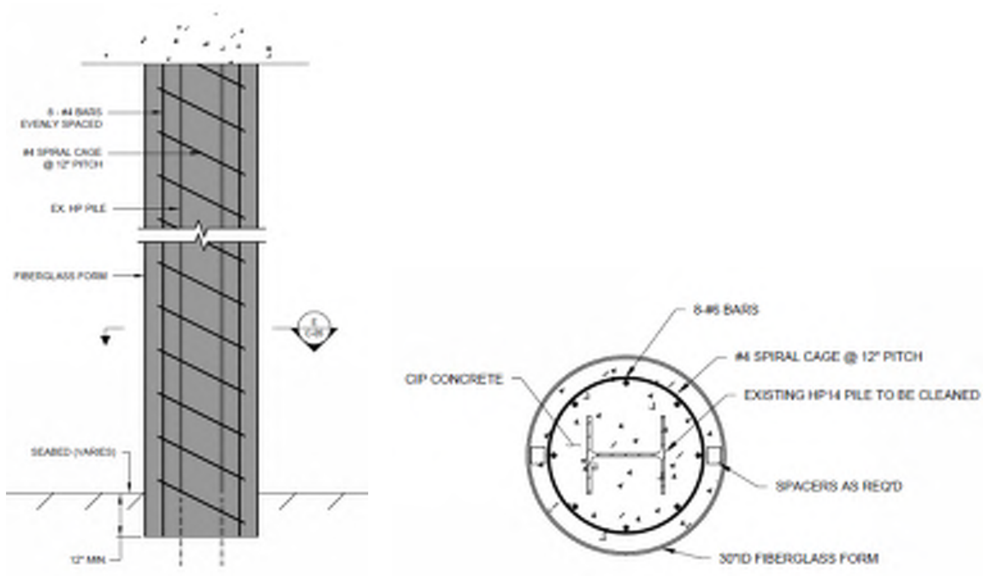

Fig. 13 – Non-Structural Pile Jacket

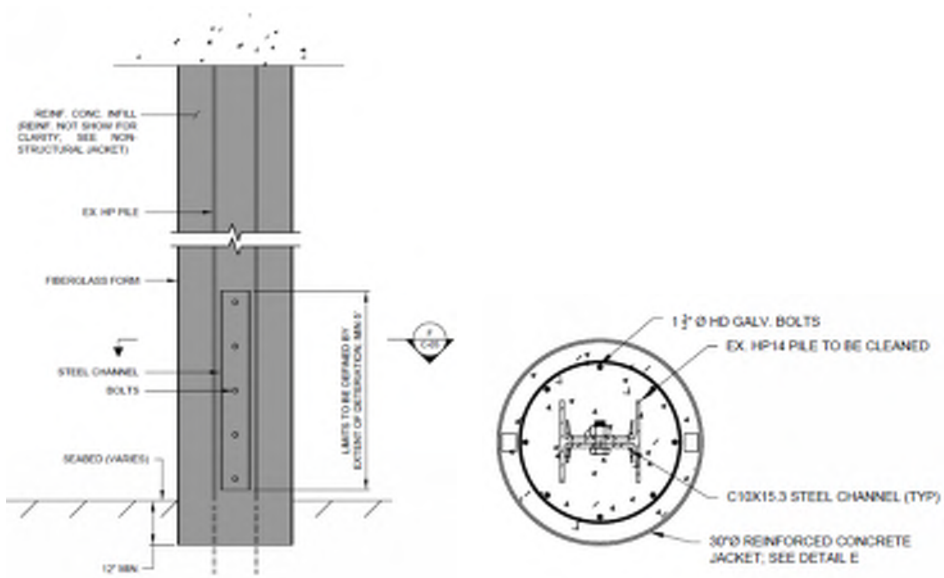

Fig. 14 – Structural Pile Jacket

4.4.2 Deck Repair Concepts

Deck repairs fall into two categories: non-structural which are preservation type repairs and structural which are rehabilitative type repairs. Preservation type repairs are shown as Fig. 15 for above deck repairs (left) and underdeck repairs (right). The repairs generally consist of chipping of unsound concrete and replacing with repair mortar. Underdeck repairs incorporate a stainless-steel welded-wire fabric and pins to support the repair mortar which is generally placed in a shotcrete manner.

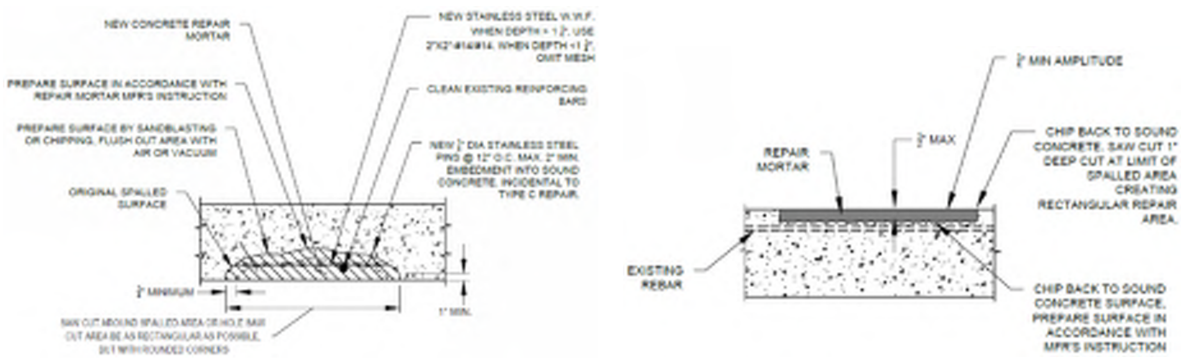

Fig. 15 – Concrete Spall and Delamination Type Repairs

A structural repair can be implemented as well in areas where the deterioration in the concrete deck has advanced to the point of loss of reinforcement section that exceeds 30%. The general approach to repair would be similar in nature to a non-structural repair. However, the repair would also include reinforcement repair as shown in Fig. 16. In areas where considerable corrosion of the reinforcement has occurred, the deteriorated areas can be removed, and new reinforcement can be spliced to the existing reinforcement by mechanical couplers or welding. The repair generally includes a welded-wire fabric to control cracking.

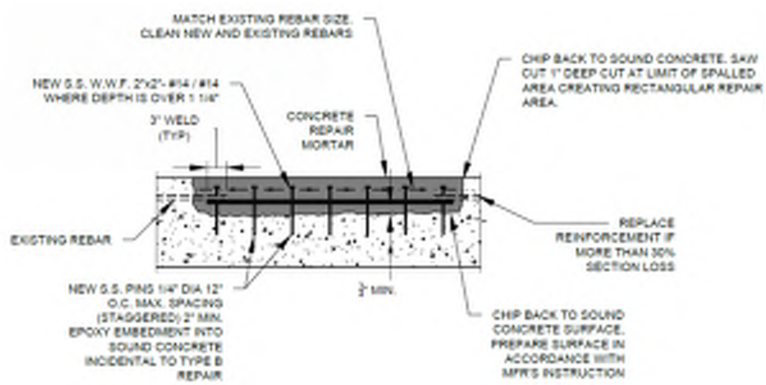

Fig. 16 – Structural Deck Reinforcement Repair

4.5 Facility Redevelopment and Marina Concept Designs

During the course of the project, status updates were provided to the Harbor Committee at regularly scheduled monthly meetings. The general consensus that has been communicated through these meetings and related discussions has been that the Town's primary interest is in redeveloping the site for future use as a recreational marina. This being the proposed future use of the site significantly alters the operational requirements of the structures from their original design intent, which was to support vehicular and pedestrian access to a side-loading ferry that would dock at the north berth. The potential layout for the future marina is an important consideration in whether to repair, replace in-kind, or remove and redevelop the structures at the site. In order to aid in these considerations, it was requested during the December 14, 2020 Harbor Committee meeting that GEI develop conceptual design layouts for a potential future marina at the site. As an extension of the inspection and assessment of the Ferry Terminal, GEI has prepared three conceptual layouts, and provided a basis of design memo describing the layout concepts. The conceptual plans and memo are provided in Appendix G for reference, and should be considered in the context of the findings of this assessment and the Town's future plans.

4.6 Cost Estimates

Cost estimates for the Long-Term and Short-Term Repair recommendations are provided in Tables 14 through 16 below. Given the varied and often difficult to estimate cost of rehabilitation work, a 20 percent contingency has been included where indicated. The unit prices were developed based on construction bids and quotes of similar recent projects. Our review observed that the pricing of pile jackets varies greatly and if this approach was pursued, we would recommend engaging with a contractor to refine the costs presented. Note that the prices do not include costs associated with design engineering or permitting of the work.

Based on the repair concepts, and scope of recommended short- and long-term repairs recommended within this memo, the following budgetary estimates are recommended:

- Short-Term Repair $5.4 million
- Long-Term Repair $12.2 million
- Total $17.6 million

Completion of the long-term repair could be expected to extend the useful life of the structure by approximately 25 years from the point of construction. Given the significant cost of rehabilitation that is necessary, consideration should also be given to a full demolition and replacement which would likely be more cost effective over the life-cycle given the significantly longer design life that would be expected.

Table 14. Short Term Repair Costs

Item	Description	Quantity	Unit	Rate	Cost
Piles	North Pier - Structural Jacket *Condition E And F Piles*	58	EA	$50,000	$2,900,000
	SOUTH PIER – STRUCTURAL JACKET *Condition E And F Piles*	31	EA	$50,000	$1,550,000
Deck	Concrete Repairs	0	SF	$250	$0
General	Public Safety	1	LS	$50,000	$50,000
				Subtotal	$4,500,000
				20% Contingency	$900,000
				Total With Contingency	**$5,400,000**

Table 15. Long Term Repair Costs

Item	Description	Quantity	Unit	Rate	Cost
Piles	North Pier – Non-Structural Jacket	98	EA	$40,000	$3,920,000
	SOUTH PIER – NON-STRUCTURAL JACKET	93	EA	$40,000	$3,720,000
Deck	Concrete Repairs	10,000	SF	$250	$2,500,000
General	Public Safety	1	LS	$50,000	$50,000
				Subtotal	$10,190,000
				20% Contingency	$2,040,000
				Total With Contingency	**$12,200,000**

We also understand the Town is evaluating alternative ways to repurpose the site. This currently includes either fully demolishing the piers, or potentially reusing select portions of the structure which were in fair condition and demolishing the remainder to install floating docks for a marina and mooring dolphins. A cost assessment for these options was performed as part of the GEI memo "Marina Conceptual Design Alternatives" and is included in Appendix G. To aid in the cost-benefit analysis, the cost to repair the existing structure is compared against other types of site improvements in the table below. For comparative purposes, we have also included replacement of the ferry terminal with a new pile supported structure of a similar footprint. The improvement costs include the necessary demolition of and repair of the existing ferry terminal and a 20 percent contingency, however, do not include the costs associated with design engineering and permitting. Relative to other site improvements, there does not appear to be a large financial benefit to repairing the structure. Repair of the structure would provide an additional 25 years of usable life, whereas a new structure could be designed for 50 to 75 years or more if desired.

Table 16. Site Improvements Cost Comparison

Site Improvement	Cost
Repair Existing Structure Entirely	$17,600,000
Marina With Mooring Dolphins (high end estimate based on Concept C)	$14,000,000
In-Kind Pier Replacement - Timber Pile and Deck	$18,300,000
In-Kind Pier Replacement - Steel Pile & Concrete Deck	$20,900,000

4.7 Conclusions

Based on GEI's analysis, it is our opinion that the existing structure has surpassed its useful life or the period when a practical repair program can be implemented. Additionally, the existing structure was purpose-built for a use that no longer exists and does not appear to be readily compatible with the Town's proposed future uses at the site. At this time, it is the opinion of GEI that the most economically and technically feasible option will be to demolish the pier in its entirety and replace the structure. The cost of replacement is in a similar order of magnitude to repair, and would provide a structure with an extended service life and design better suited to the future uses. This is a significant financial investment and the design of the replacement structure should be carefully considered to maximize the potential usefulness, compatibility with future uses, and economic benefit of the project.

Appendix A

References

Reference No.	Folder* / Source	File	Year	Description
1	3 Electrical Service	13. Three Electrical Services 1990	1991	Electrical work performed by Electrical Services Inc.
2	Access Ramp Recoating 1995	17. Division 9 finishes Section 1 -15	--	Portion of contract regarding access ramp
3	Access Ramp Recoating 1995	17. Environmental Assessment Review Process	--	Portion of contract regarding access ramp
4	Access Ramp Recoating 1995	17. Inspection for Access Ramp1996	1996	East and west access ramp upgrades
5	Access Ramp Recoating 1995	17. Pay Reg Engineering Services 6.1995 to 02.1996	1996	West access ramp upgrades
6	Access Ramp Recoating 1995	17. Reg Engineering Services 11.1995 to 07.1996	1996	East access ramp upgrades
7	Access Ramp Recoating 1995	17. Spec - East Ramp Support	1996	East access ramp support structure replacement
8	Access Ramp Recoating 1995	17. Spec Marine Fender System Replacement	1995	Marin fender system replacement
9	Access Ramp Support Structure 1996	Access Ramp Support St. 01.01.96 - 12.12.96	1996	East access ramp structural upgrades
10	ALL Full Size Plans Scanned	A. Steel Framing for Adj Car Ramp 08.1954	1954	Drawings of vehicle access ramp steel framing structure

Reference No.	Folder* / Source	File	Year	Description
11	ALL Full Size Plans Scanned	A. Terminal Pier plan sections 05.1954	1954	Drawings of pier structure
12	ALL Full Size Plans Scanned	A. Terminal Anchor Bolt Plan 08.1954	1954	Drawings of anchor bolt plan
13	ALL Full Size Plans Scanned	A. Terminal Boring Date 05.1954	1954	Drawings showing boring data from site
14	ALL Full Size Plans Scanned	A. Terminal General Plan 05.1954	1954	Plan view of waterfront facility
15	ALL Full Size Plans Scanned	A. Terminal General Plan 05. 1954.pdf	1954	Plan view of waterfront facility
16	ALL Full Size Plans Scanned	B. Terminal Water & Fuel detail 01.1955	1955	General plan of water & fuel infrastructure within facility
17	ALL Full Size Plans Scanned	B. Terminal Water & Fuel Piping 01.1955	1955	Details of water & fuel infrastructure within facility
18	ALL Full Size Plans Scanned	C. Terminal Apron, pier and trestle pile record 01.1956	1956	Apron, pier, and trestle pile records
19	ALL Full Size Plans Scanned	D. Terminal I Vehicle Ramp Modi Concrete 12.30.82	1982	Drawings of vehicle ramp modifications
20	ALL Full Size Plans Scanned	D. Terminal Utility Plan 09.22.82	1982	Plan view of pier utility infrastructure
21	ALL Full Size Plans Scanned	D. Terminal Vehicle Ramp Modi. Steel Plan 12.30.82	1982	Drawings of vehicle ramp modifications
22	ALL Full Size Plans Scanned	D. Terminal Pedestrian Ramp, Pier Modi 12.30.82	1982	Drawings of pedestrian ramp
23	ALL Full Size Plans Scanned	E. Completion of Terminal Renovations 12.09.84	1984	Drawings of east access ramp
24	ALL Full Size Plans Scanned	F. Site plan as built Argentina, Newfoundland Dock 12.02.88	1988	As built drawings of waterfront structure in Newfoundland.
25	ALL Full Size Plans Scanned	G. Terminal Catwalk Plan and details 1.17.89	1989	Drawings of catwalk structure

Reference No.	Folder* / Source	File	Year	Description
26	ALL Full Size Plans Scanned	G. Terminal Standoff Dolphin Plan & details 02.09.1989	1989	Drawings of stand-off dolphin structure
27	ALL Full Size Plans Scanned	G. Terminal Warping Dolphin 02.09.89	1989	Drawings of warping dolphin structure
28	ALL Full Size Plans Scanned	H. Marine Fender DETAILS 09.1990	1990	Details of marine fender system upgrades
29	ALL Full Size Plans Scanned	H. Marine Fender plans, elevation and side view 09.1990	1990	Plan view of Marine fender system upgrades
30	ALL Full Size Plans Scanned	I. Marine Fender Elevation and side view 04.04.94	1994	Elevation view of fender system upgrades
31	ALL Full Size Plans Scanned	J. Marine Fender Existing and new fender panels 04.04.95	1995	Plan view of fender system upgrades
32	ALL Full Size Plans Scanned	J. East Ramp Support Structure 10.10.95	1995	Details of east access ramp support structure
33	ALL Full Size Plans Scanned	J. Framing Plan and Grating Layout & details 10.10.95	1995	Plan view of proposed fender system upgrades
34	ALL Full Size Plans Scanned	J. Marine Fender Details 04.04.95	1995	Details of proposed fender system upgrades
35	ALL Full Size Plans Scanned	J. Marine Fender Replacement 04.04.95	1995	Details of proposed fender system upgrades
36	ALL Full Size Plans Scanned	J. Proposed dock extension Site Plan 11.14.95	1995	Plan view of proposed pier modifications
37	ALL Full Size Plans Scanned	J. Terminal Proposed Dock Ext 11.14.95	1995	Plan view of proposed pier modifications
38	ALL Full Size Plans Scanned	K. 2 Borden Steel Heavy Duty Grating. East Ram 05.08.96	1996	Plan view of proposed east access ramp upgrades
39	Bar Harbor Dock Transfer Bridge & platform	Ships Gangway Platform Jan 1985	1985	Inspection of renovations to ferry terminal

Reference No.	Folder* / Source	File	Year	Description
40	Bar Harbor Dock Transfer Bridge & platform	Transfer Bridge 04. 1989	1989	Note from Mr. O.P. Tinker, Terminal Manager, regarding access ramp weight limitations
41	Bar Harbor Waterline Replacement	12. Bar Harbor Waterline replace 5.13.88	1996	Observed leak in water main
42	Building Renovations	8. Building Renovation	1984	Terminal building renovations
43	Building Renovation 1984\Sliding Glass Door Renovation	Building Renovation October 1984	1984	Repeat of file "8. Building Renovation" in folder "Building Renovations"
44	Building Renovation 1984\Sliding Glass Door Renovation	Renovation Feb 1984 Sliding Glass Door Plan	1984	Terminal building renovations
45	Cruise Ship Modifications	14. Modification of Cruise Ships	1995	Proposed modifications to facility including extension of pier
46	Fendering	3. Fendering Folder Feb 14 91 to Dec 1991	1991	Upgrades to fendering system
47	Fendering	09.01.1987 Warping Dolphin Existing Condition Report	--	Unable to open document
48	Fendering	Fender Plan Inspection Arrangement 7.19.1988	1988	Handwritten notes from fender inspection on plan view drawing
49	Fendering	Material Plan Sch March 1991	1991	Mostly illegible drawing titled "Piles & Miscellaneous"
50	Fendering	Plans Fender Replace and Elevation 09.01.1990	1990	Elevation view of fendering system
51	Fendering	Plans Fender Replacement 09.01.1990	1991	Drawings pertaining to upgrades to fendering system
52	Fendering	1 Fendering Folder Oct 90 to Nov 1990	1991	Upgrades to fendering system
53	Fendering	2. Fendering folder Nov 90 to Feb 7 1991	1991	Upgrades to fendering system
54	Ferry Terminal Renovation	8. Ferry Terminal Renovation Plans 1982 mark up	1982	Plan view drawing of pier

Reference No.	Folder* / Source	File	Year	Description
55	Ferry Terminal Renovation	8. Ferry Terminal Renovation Plans 1984 . Original	1984	Ferry terminal renovations
56	Gangway 1988 - Telescoping	11. Telescoping Gangway 1988	1986	Upgrades to gangway system
57	Geo Technical Investigation 04.21.95 to 07.10.95	4. Geo Technical Investigation 04.21.95 to 07.10.95	1995	Geotechnical investigation
58	Marine Fender System Replacement	18. Fender Panel Details 07.1995	1995	Marked up details of fender panels
59	Marine Fender System Replacement	18. Prock Marine Documents	1995	Specifications of fender pile replacement structures
60	Marine Fender System Replacement	18. Proposed Fender Panel 12.12.94	1994	Plan view drawing of proposed new fendering panels
61	Marine Fender System Replacement	18. Capital Project Completion Reports 02.06.96	1996	Documents related to fendering system upgrades, geotechnical upgrades, and pier pile cap concrete repairs
62	Marine Fender System Replacement	18. Fender Panel Pile Elevations 12.13.91 to 07.16.96	1996	Fender pile inspection and replacement
63	Marine Qualification	14. Marine Qualification	1995	Appledore Engineering qualifications submittal
64	Misc. File	9. 1994 - Misc. File	--	Proposed upgrades to facility (1994). Information regarding disposal of PCBs (1987). Proposed maintenance to concrete on facility (1985)/
65	Passenger Walkway and Canopy	5. Passenger Walkway Canopy Damage to Repair 09.25.95 to 05.03.96	1996	Repairs to damaged passenger walkway canopy
66	Passenger Walkway Walkway and Canopy	10. Passenger Walkway Lifting Folder System	1992	Passenger walkway operating rope lifting system maintenance

Reference No.	Folder* / Source	File	Year	Description
67	Passenger Walkway Walkway and Canopy	10. Passenger walkway lifting System Plan	Unclear	Details and section of ramp building lifting mechanism
68	Passenger Walkway Walkway and Canopy	11. Preliminary Concept of Passenger Assess 1985	1995	Passenger access ramp drawings
69	Pier Cap Concrete Restoration	15. Pier Cap Concrete Restoration 05.10.95 to 10.18.95	1995	Pier concrete cap repairs on northwest and south side of pier
70	Ship Gangway Platform and Transfer Bridge	6. Ships Gangway Platform Jan 1985	1985	Copy of “6. Ships Gangway Platform Jan 1985” in folder "Bar Harbor Dock Transfer Bridge & platform"
71	Ship Gangway Platform and Transfer Bridge	7. Transfer Bridge 04. 1989	189	Copy of “7. Transfer Bridge 04. 1989” in folder "Bar Harbor Dock Transfer Bridge & platform"
72	Prock Marine Company Files	Ferry Terminal Inspection – Marenco 2007	2007	Structural inspection of pier facility
73	Prock Marine Company Files	Ferry Terminal Structural Analysis – Eastpoint Engineering 2008	2008	Structural inspection of pier facility
74	Prock Marine Company Files	Scanned from Prock Marine	Varies	Documents provided by Prock Marine
75	Town Website	Phase One - Feasibility Study for the Acquisition of the Bar Harbor Ferry Terminal	2011	Description in file name
76	Town Website	Phase Two – Progress Report – May 31, 2012	2012	Description in file name
77	Town Website	Phase Two – Final Report – August 2012	2012	Description in file name
78	Town Website	Bar Harbor Ferry Terminal Progress Report presented March 10, 2016	2016	Description in file name

Reference No.	Folder* / Source	File	Year	Description
79	Town Website	Bar Harbor Ferry Terminal Cohesive Acquisition & Development Strategy March 14, 2017	2017	Description in file name
80	Town Website	4 Ways To Vote for Article 12 and Citizen Initiative 13	Not dated	Description in file name
81	Town Website	Town Attorney letter - Proposed Land Use Amendments #12 and #13 Conflicts 05/23/2017	2017	Description in file name
82	Town Website	MDOT Response to Use of Bond Funds at Ferry Terminal 06/05/2017	2017	Description in file name
83	Town Website	July 17, 2017 Ferry Terminal Visioning Workshop - Presentation with community comments from the workshop incorporated	2017	Description in file name
84	Town Website	Ferry Terminal Property Advisory Committee Recommendations to Council November 14, 2017	2017	Description in file name
85	Town Website	CES Project Operations and Maintenance Cost Estimates - supplement to Business Plan May 14, 2018	2018	Description in file name
86	Town Website	CES Capital Cost Estimates - supplement to Business Plan May 14, 2018	2018	Description in file name
87	Town Website	Ferry Terminal Property Proposed Business Plan, May 14, 2018 presentation	2018	Description in file name

Reference No.	Folder* / Source	File	Year	Description
88	Town Website	Ferry Property Financial Model FINAL May 23, 2018	2018	Description in file name
89	Town Website	Ferry Property Business Plan FINAL June 6, 2018	2018	Description in file name
90	Town Website	Bay Ferries Proposal to the Town of Bar Harbor July 11, 2018	2018	Description in file name
91	Town Website	Steering Committee Recommendations February 11, 2019	2019	Description in file name
92	Cianbro Files	FERRY TERMINAL REPAIR PLANS, TECT ASSOCIATES, 2018	2018	Plans for installation of RO-RO Ramp, fendering upgrades, and pile repairs on north pier.
93	TEC Associates Files	“Bar Harbor Drawings”	Varies	Plans for original Ferry Terminal construction and various modifications (155 sheets total)

*Folder” refers to the electronic document organization for the files provided by the Town during the RFP process.

Appendix B

Inspection and Analysis Figures

PRELIMINARY

ELEVATION	MLLW*	NAVD88
BFE	+22.97	+17.00
HAT	+13.31	+7.34
MHHW	+11.37	+5.40
MHW	+10.94	+4.97
MSL	+5.67	-0.30
MLW	+0.38	-5.59
MLLW	0.00	-5.97

*PROJECT DATUM

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SCALE: 1" = 20'

Attention:

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If this scale bar does not measure 1" then drawing is not original scale.

DRAFT

Designed:	---
Drawn:	DAP
Checked:	DJB
Approved:	---
P.E. No:	---
GEI Project	2004148

GEI Consultants

GEI CONSULTANTS, INC.
5 MILK STREET
PORTLAND, ME 04101
(207)797-8901

Town of Bar Harbor
93 Cottage St.
Bar Harbor, ME 04609

Ferry Terminal Inspection & Assessment

121 Eden St.
Bar Harbor, ME

NO	DATE	ISSUE/REVISION	APP
3	2/8/2021	FINAL REPORT	DJB
2	11/13/2020	TASK 4 DRAFT SUBMITTAL	DJB
1	9/15/2020	PROJECT KICKOFF	DJB

SHEET NAME: PIER SCHEMATIC PLAN

SHEET NO.: C-01

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PRELIMINARY

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MLW	+0.38	-5.59
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Checked: DJB
Approved: ---
P.E. No: ---
GEI Project 2004148

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Town of Bar Harbor
93 Cottage St.
Bar Harbor, ME 04609

Ferry Terminal Inspection & Assessment

121 Eden St.
Bar Harbor, ME

NO	DATE	ISSUE/REVISION	APP
3	2/8/2021	FINAL REPORT	DJB
2	11/13/2020	TASK 4 DRAFT SUBMITTAL	DJB
1	9/15/2020	PROJECT KICKOFF	DJB

SHEET NAME
STEEL PILE CONDITION ASSESSMENT RATINGS

SHEET NO.
C-02

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MLW	+0.38	-5.59
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*PROJECT DATUM

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Approved: ---
P.E. No: ---
GEI Project 2004148

GEI Consultants
GEI CONSULTANTS, INC.
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PORTLAND, ME 04101
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Town of Bar Harbor
93 Cottage St.
Bar Harbor, ME 04609

Ferry Terminal Inspection & Assessment

121 Eden St.
Bar Harbor, ME

NO	DATE	ISSUE/REVISION	APP
3	2/8/2021	FINAL REPORT	DJB
2	11/13/2020	TASK 4 DRAFT SUBMITTAL	DJB
1	9/15/2020	PROJECT KICKOFF	DJB

SHEET NAME
SUPERSTRUCTURE CONDITIONS

SHEET NO.
C-03

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*PROJECT DATUM

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Checked: DJB
Approved: ---
P.E. No: ---
GEI Project 2004148

GEI Consultants
GEI CONSULTANTS, INC.
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93 Cottage St.
Bar Harbor, ME 04609

Ferry Terminal Inspection & Assessment

121 Eden St.
Bar Harbor, ME

NO	DATE	ISSUE/REVISION	APP
1	2/8/2021	FINAL REPORT	DJB

SHEET NAME
GRAVITY LOAD STRUCTURAL ASSESSMENT PILE AND DECK LIMITATIONS

SHEET NO.
C-06

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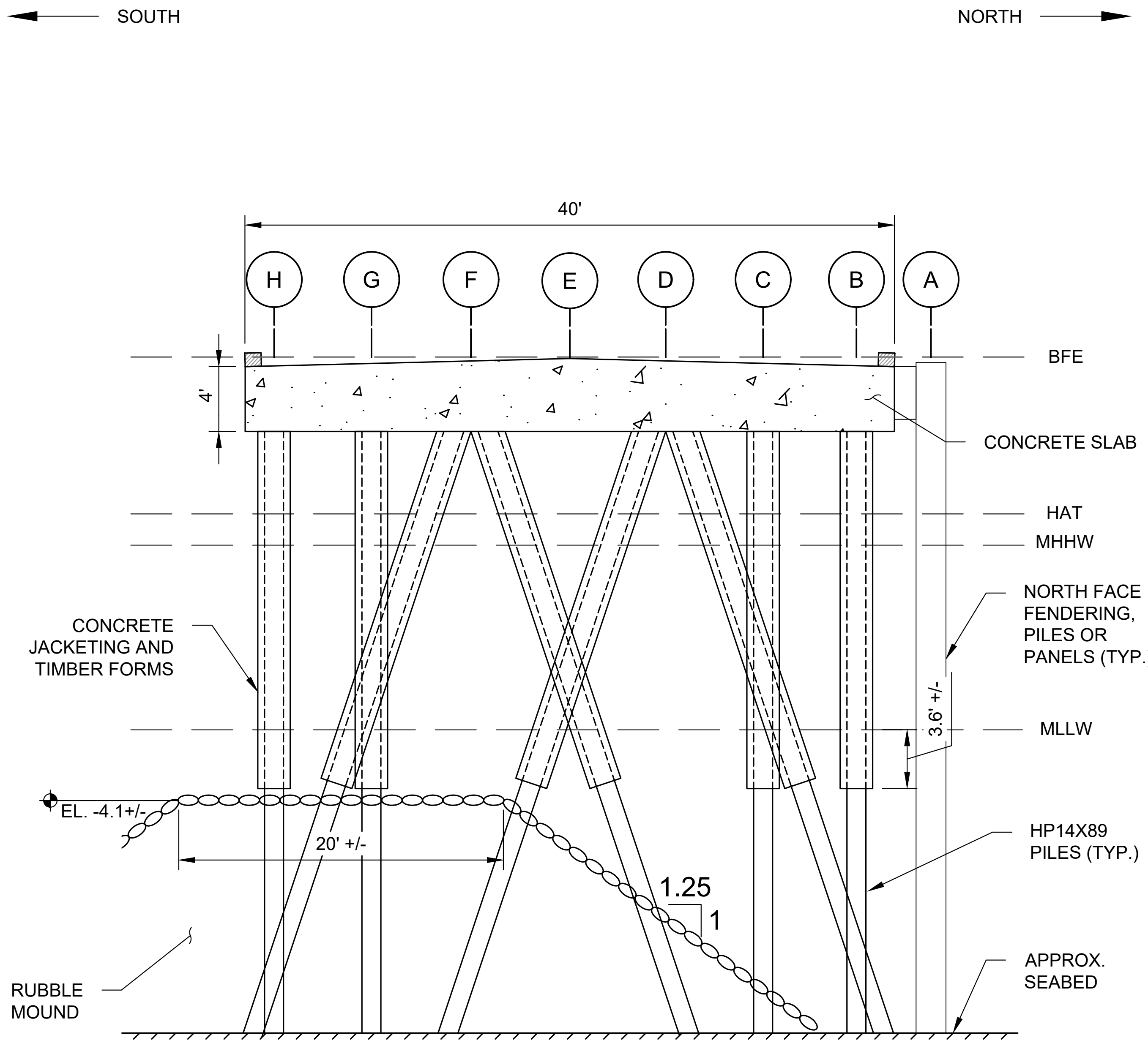

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SCALE: 1" = 10'

NOTES:

1. NORTH FACE FENDER SYSTEM NOT INCLUDED IN INSPECTION

Ferry Terminal Inspection & Assessment 121 Eden Street, Bar Harbor, Maine	GEI Consultants	TYPICAL PIER SECTIONS BENTS P2, P4, P17 & P19
Town of Bar Harbor Bar Harbor, Maine	Project 2004148	FEBRUARY, 2021 S-01

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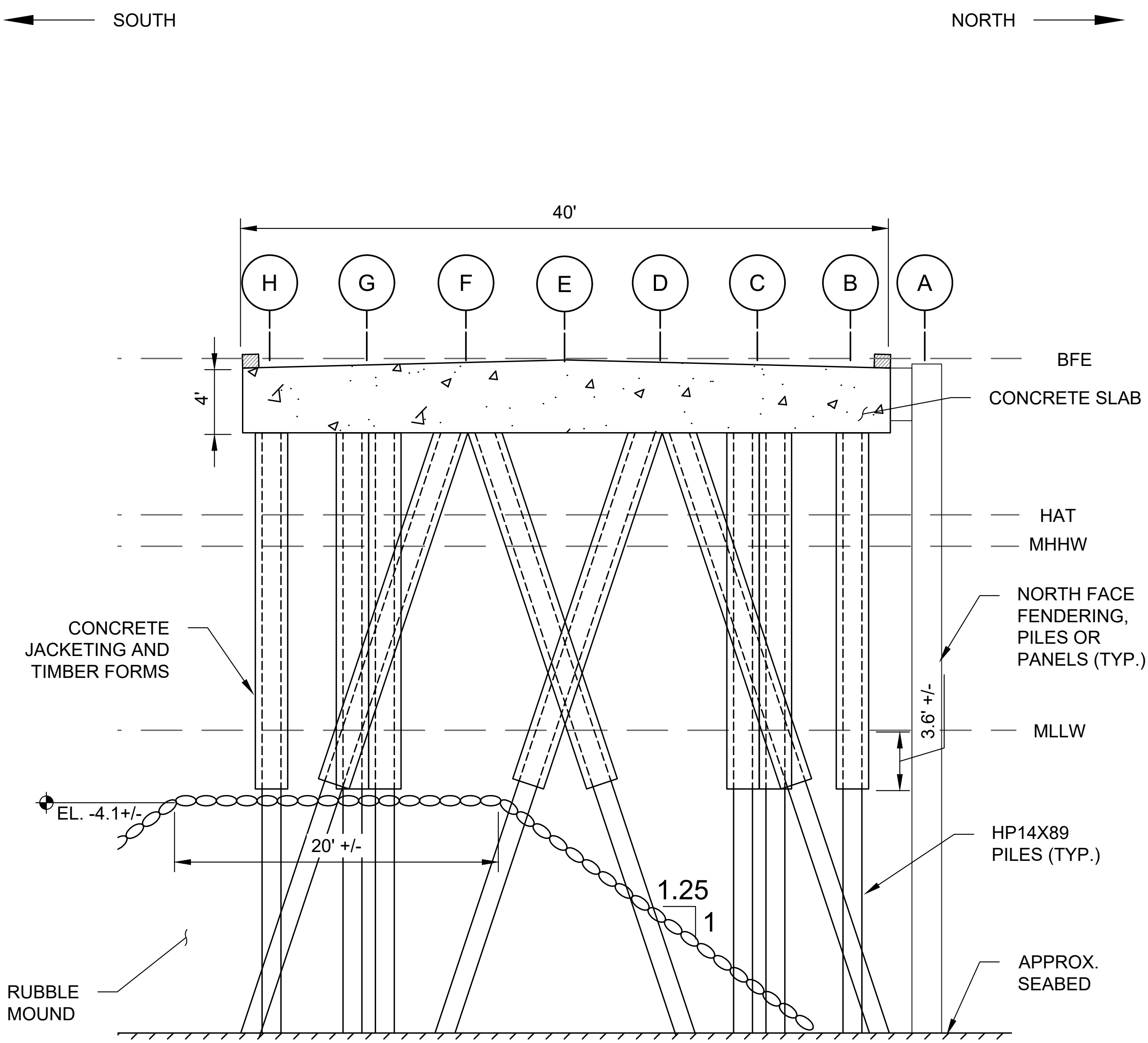

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MLW	+0.38'
MLLW	0.0'

NOTES:

1. NORTH FACE FENDER SYSTEM NOT INCLUDED IN INSPECTION

0 10 20

SCALE: 1" = 10'

Ferry Terminal Inspection & Assessment
121 Eden Street, Bar Harbor, Maine

Town of Bar Harbor
Bar Harbor, Maine

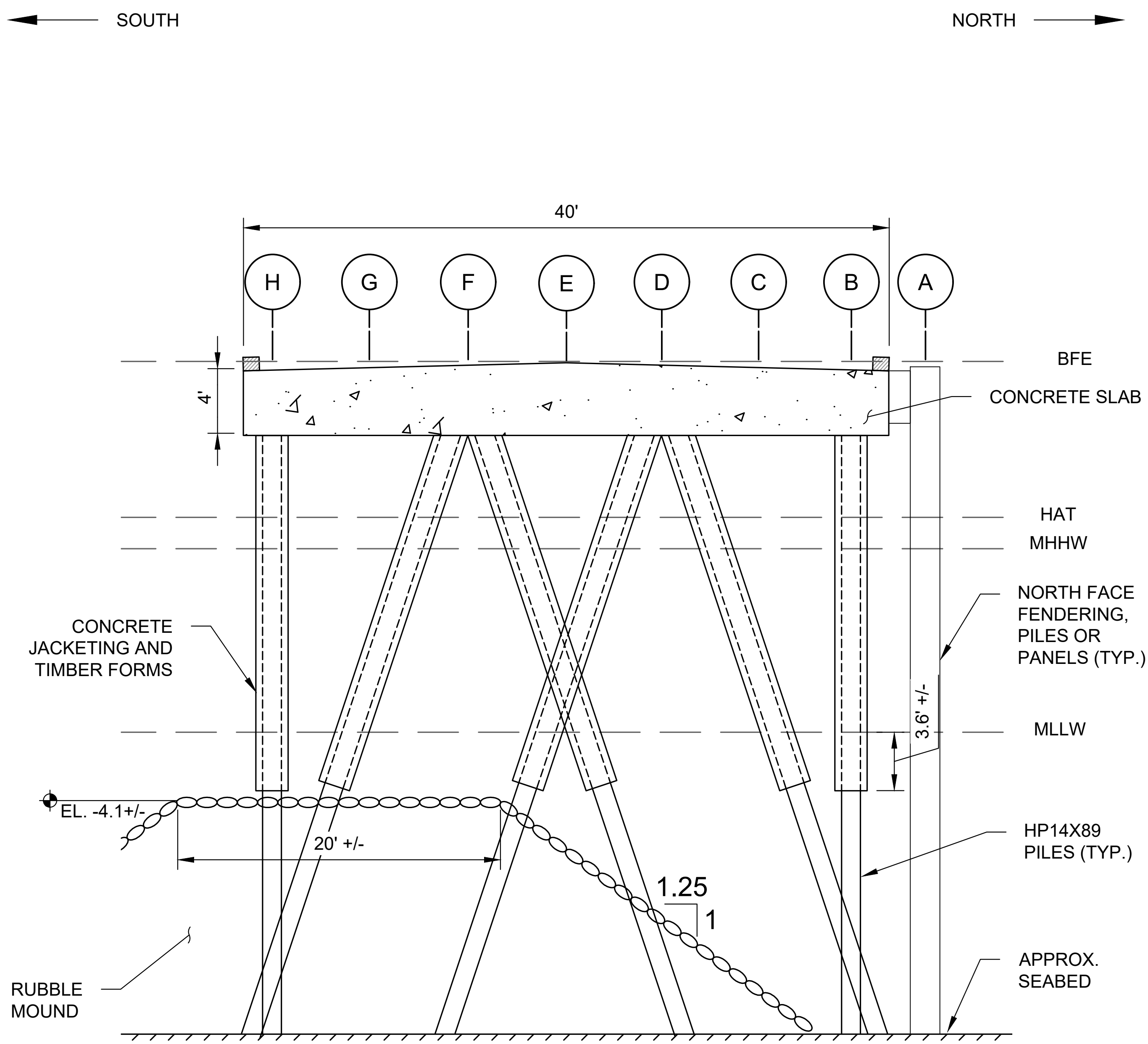
GEI Consultants

Project 2004148

TYPICAL PIER SECTIONS
BENTS P3 & P18

FEBRUARY, 2021

S-02



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MHW	+10.94'
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MLLW	0.0'

NOTES:

1. NORTH FACE FENDER SYSTEM NOT INCLUDED IN INSPECTION

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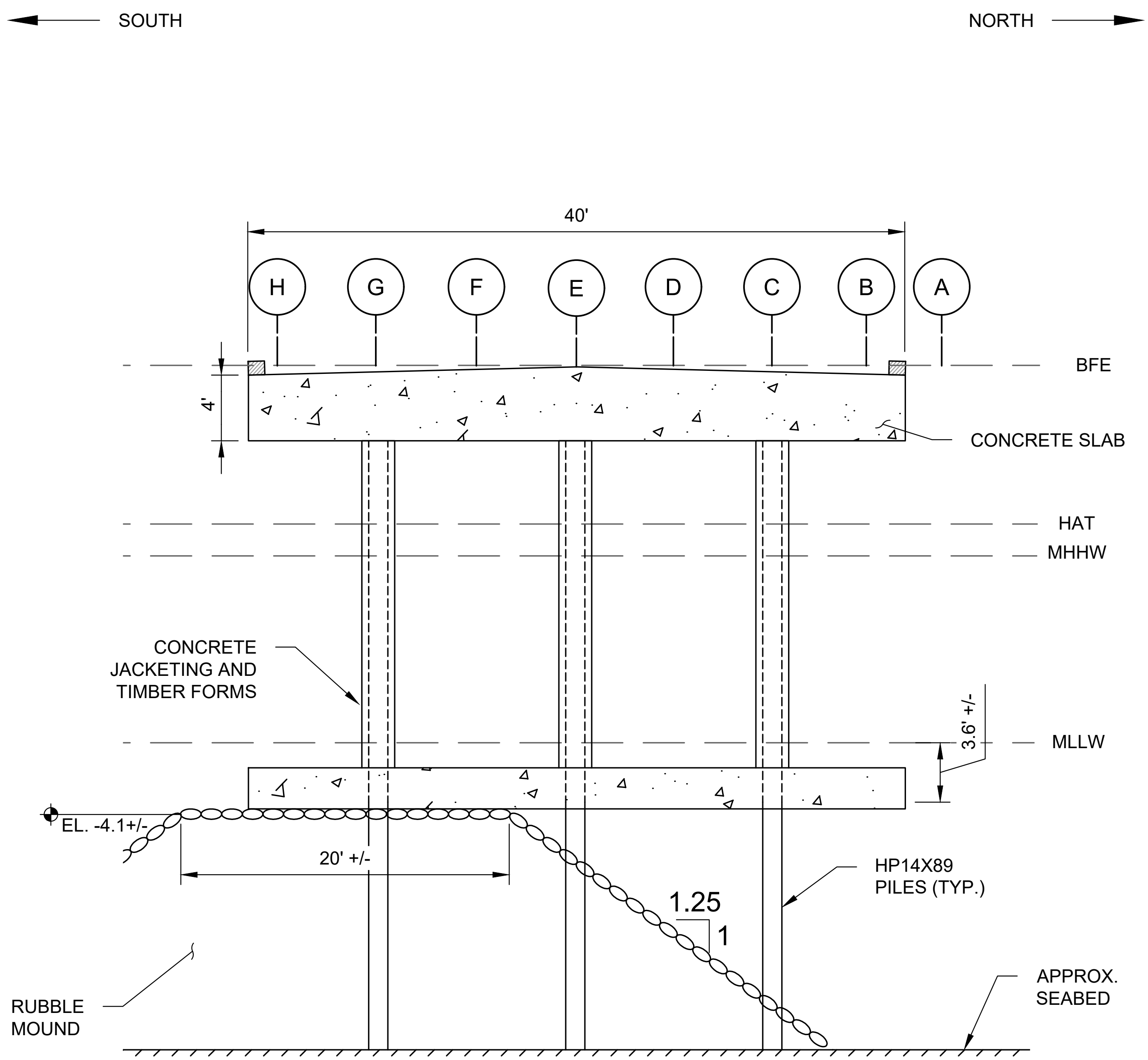
SCALE: 1" = 10'

Ferry Terminal Inspection & Assessment 121 Eden Street, Bar Harbor, Maine	GEI Consultants	TYPICAL PIER SECTIONS BENTS P1, P5-P10, P13-P16 & P20
Town of Bar Harbor Bar Harbor, Maine	Project 2004148	FEBRUARY, 2021 S-03



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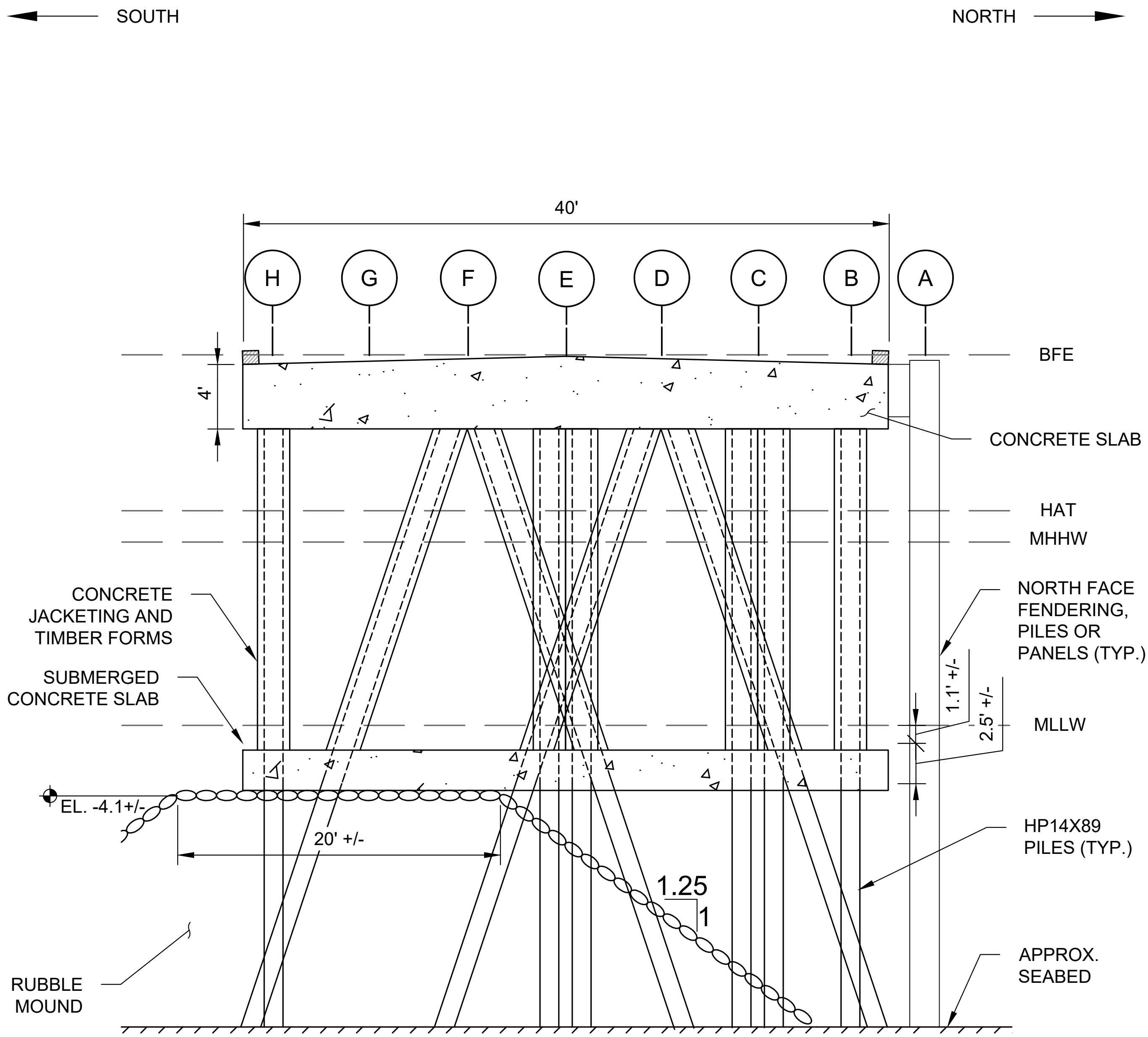
MLLW	0.0'

NOTES:

1. NORTH FACE FENDER SYSTEM NOT INCLUDED IN INSPECTION

Ferry Terminal Inspection & Assessment 121 Eden Street, Bar Harbor, Maine	GEI Consultants	TYPICAL PIER SECTIONS BENTS P5A, P5B, P15A, P15B
Town of Bar Harbor Bar Harbor, Maine	Project 2004148	FEBRUARY, 2021 S-04



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MHW	+10.94'
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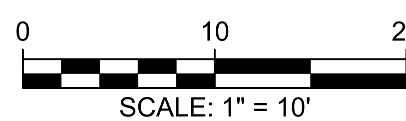

NOTES:

1. NORTH FACE FENDER SYSTEM NOT INCLUDED IN INSPECTION

Ferry Terminal Inspection & Assessment
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TYPICAL PIER SECTIONS
BENTS P11 & P12

FEBRUARY, 2021

S-05

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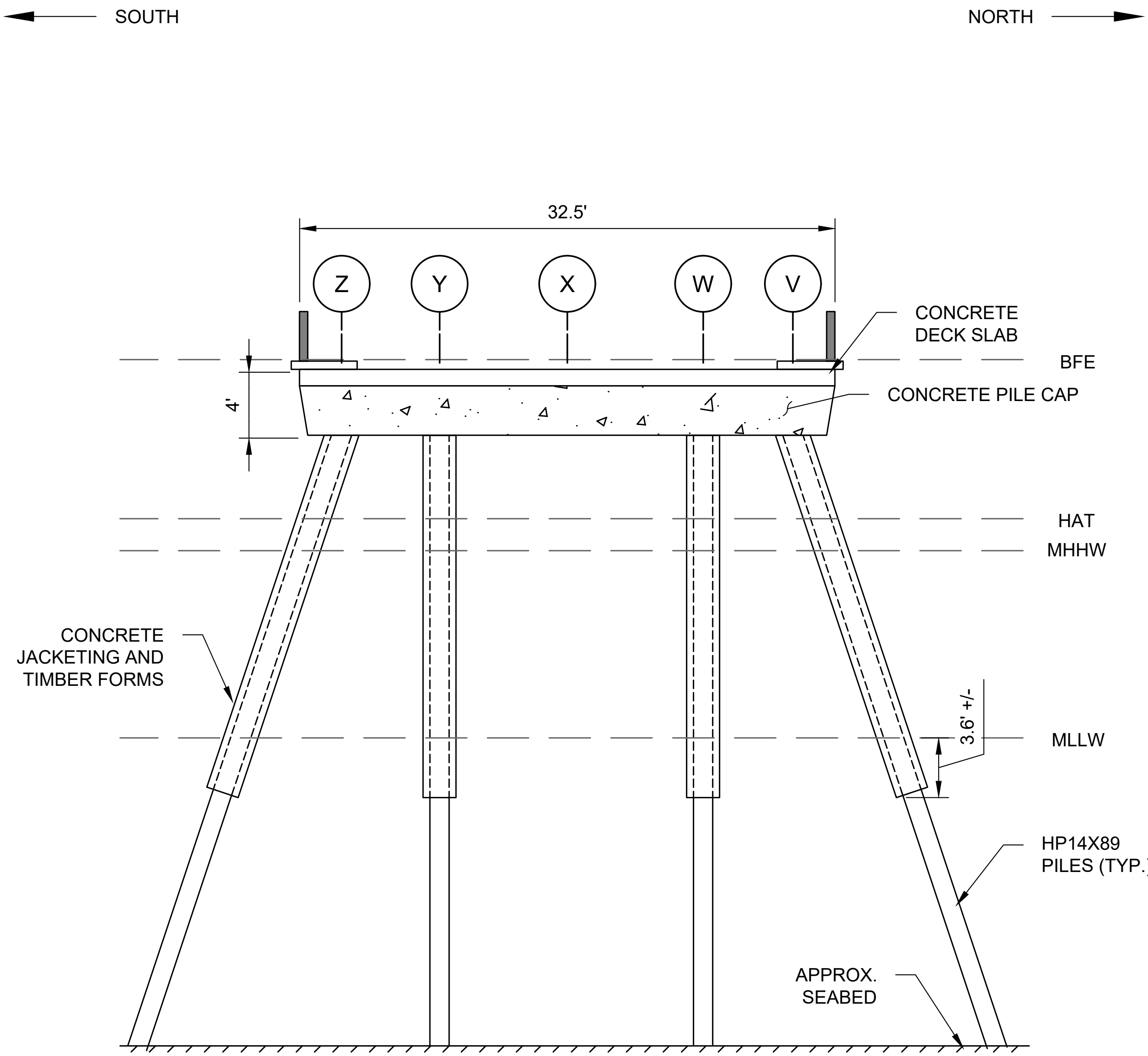

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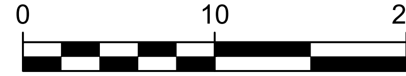
HAT	+13.31'
MHHW	+11.37'
MHW	+10.94'
MLW	+0.38'
MLLW	0.0'



SCALE: 1" = 10'

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TYPICAL PIER SECTIONS
BENTS 1-8, 17-23

FEBRUARY, 2021

S-06

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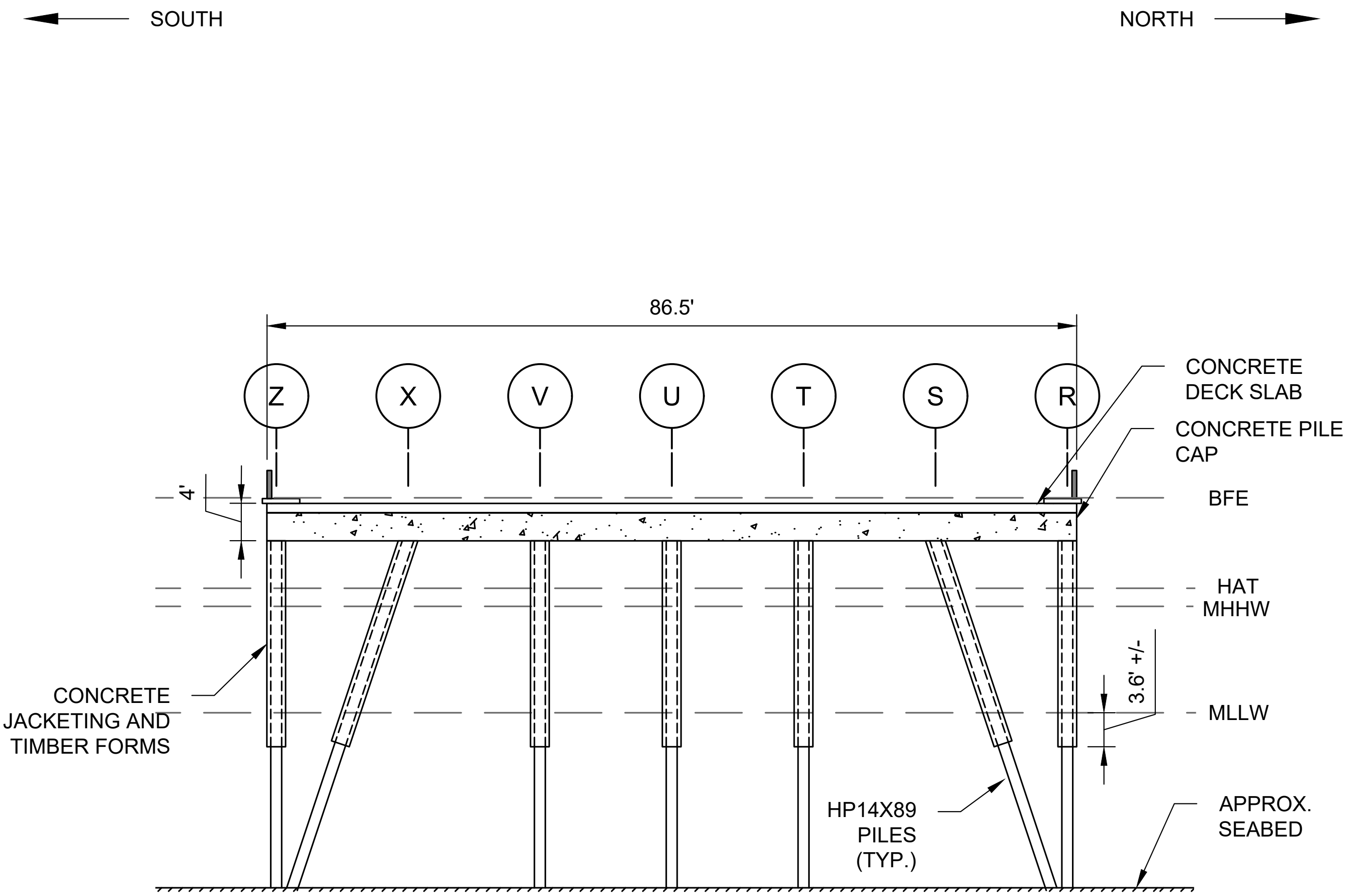

BFE	+22.97'
HAT	+13.31'
MHHW	+11.37'
MHW	+10.94'
MLW	+0.38'
MLLW	0.0'

0 20 40
SCALE: 1" = 20'

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TYPICAL PIER SECTIONS
BENTS 9-16

FEBRUARY, 2021

S-07

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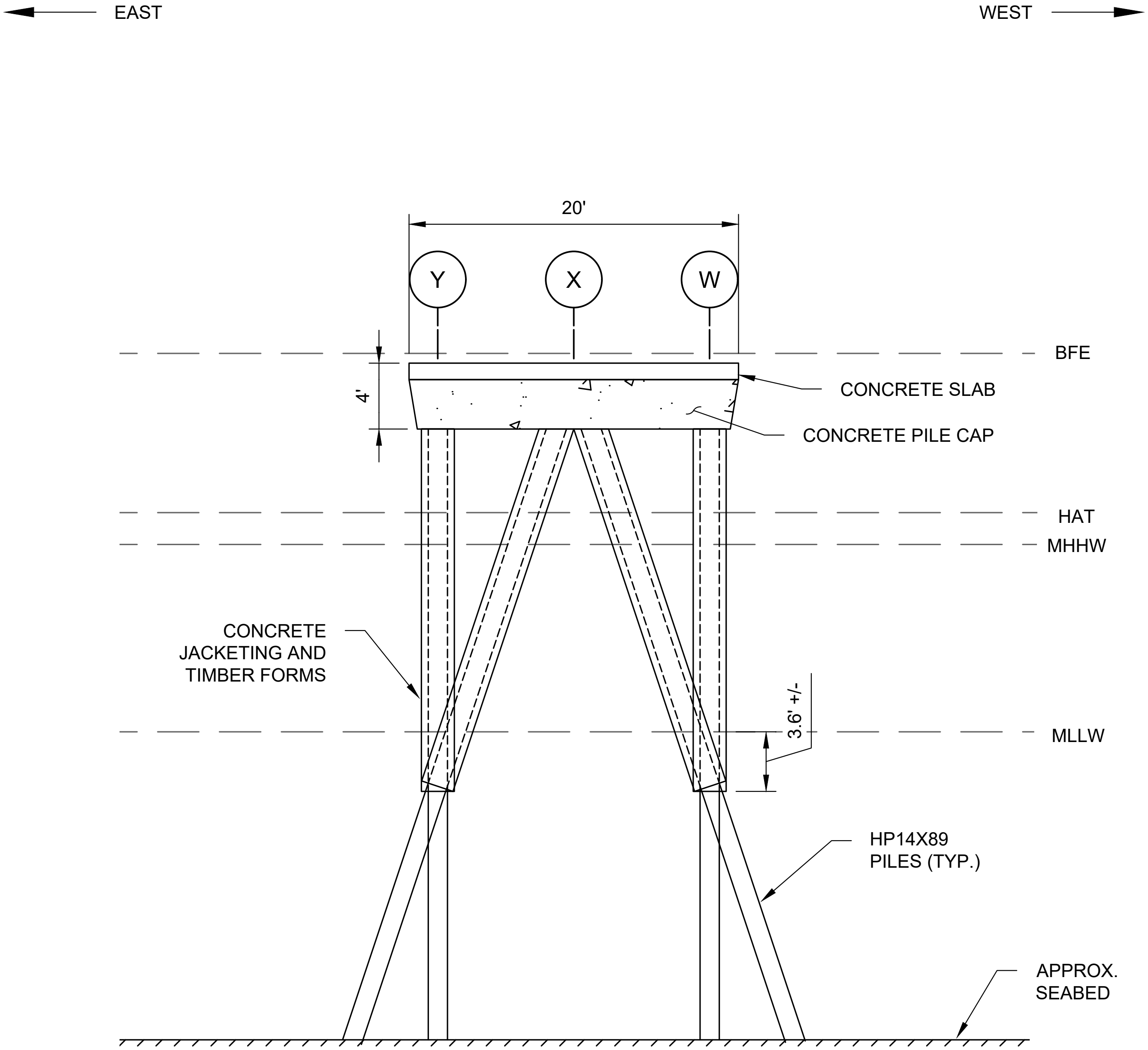

BFE	+22.97'
HAT	+13.31'
MHHW	+11.37'
MHW	+10.94'
MLW	+0.38'
MLLW	0.0'

0 10 20

SCALE: 1" = 10'

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TYPICAL PIER SECTIONS
BENTS 24 & 25

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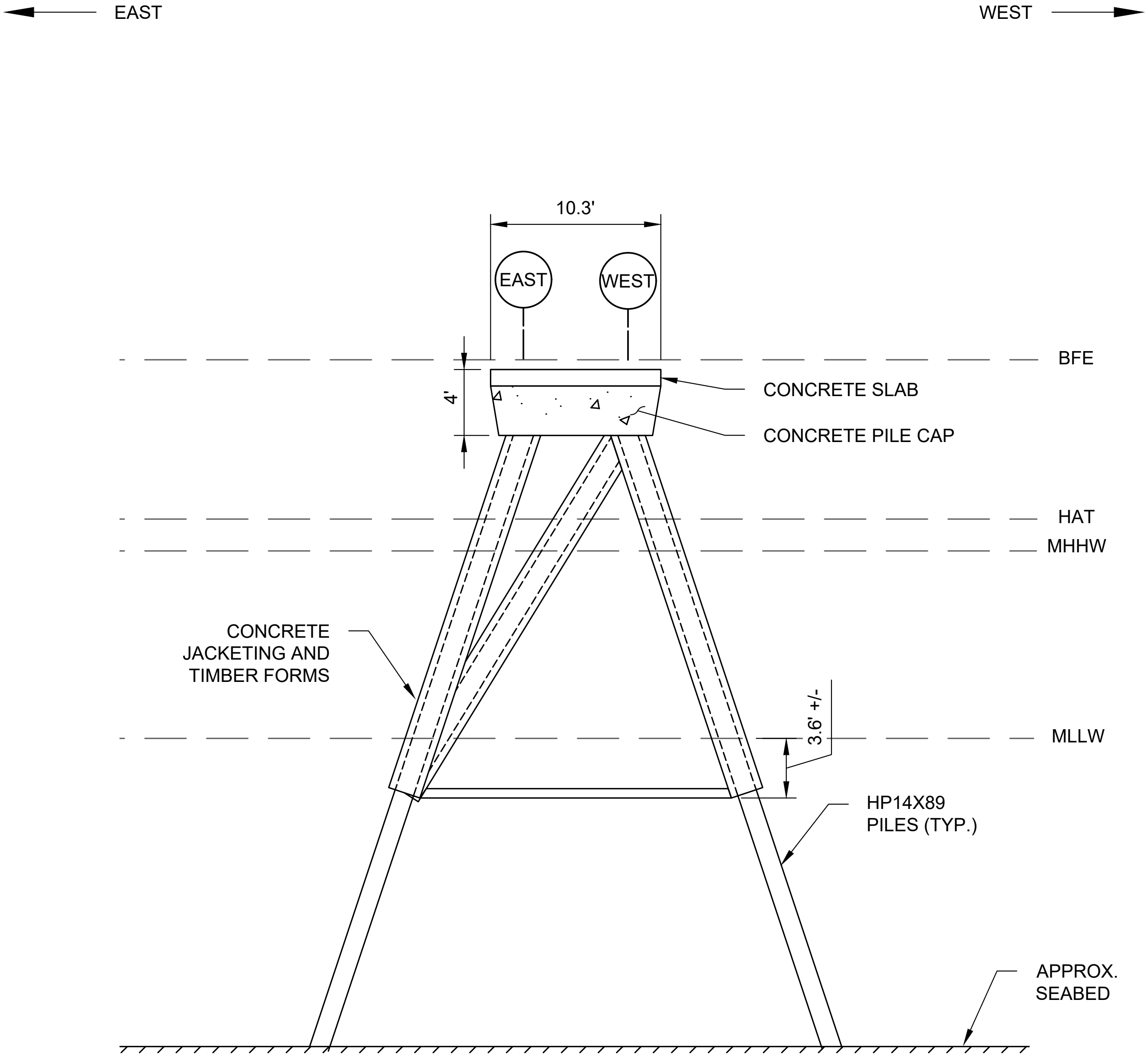

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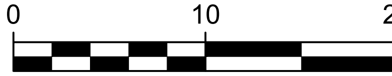
HAT	+13.31'
MHHW	+11.37'
MHW	+10.94'
MLW	+0.38'
MLLW	0.0'



SCALE: 1" = 10'

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TYPICAL PIER SECTIONS
BENT 26

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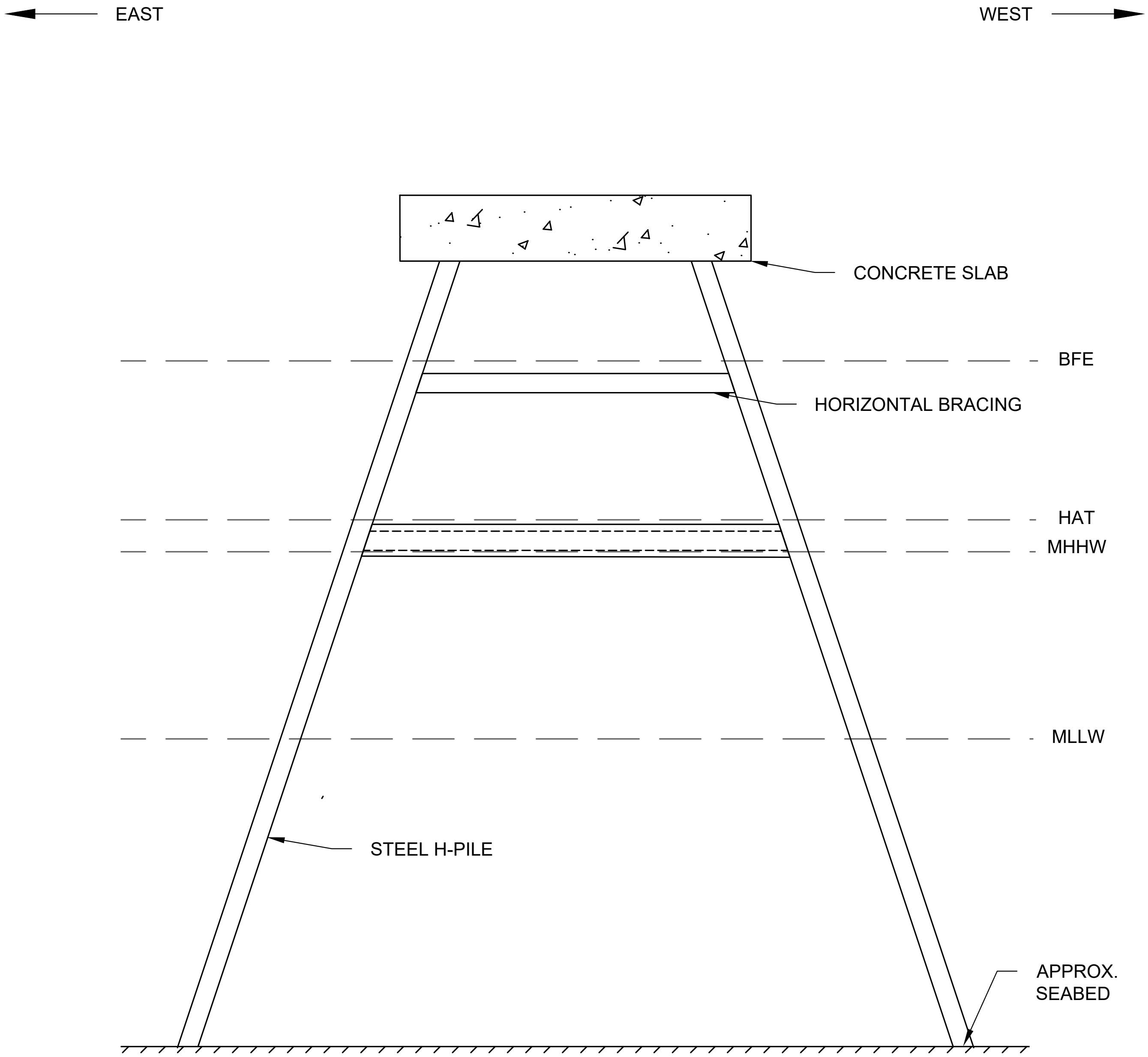

BFE	+22.97'
HAT	+13.31'
MHHW	+11.37'
MHW	+10.94'
MLW	+0.38'
MLLW	0.0'

0 10 20

SCALE: 1" = 10'

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TYPICAL PIER SECTIONS
BENT 27

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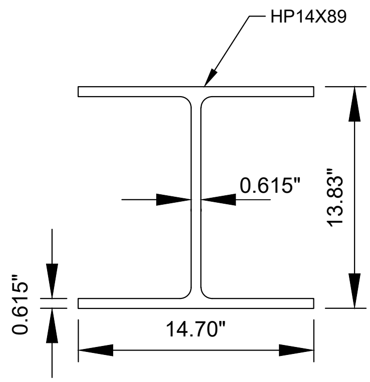
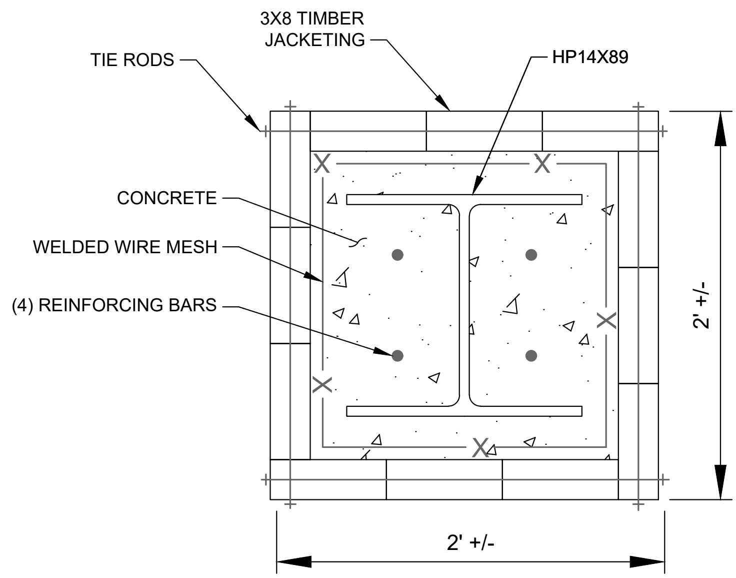
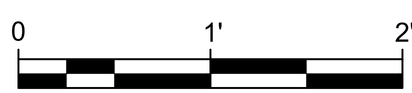


TYPICAL H-PILE SECTION



TYPICAL H-PILE SECTION WITH CONCRETE ENCASEMENT AND TIMBER JACKETING



SCALE: 1" = 1'

Ferry Terminal Inspection & Assessment 121 Eden Street, Bar Harbor, Maine	GEI Consultants	TYPICAL H-PILE SECTIONS
Town of Bar Harbor Bar Harbor, Maine	Project 2004148	FEBRUARY, 2021 S-11

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Appendix C

Inspection Tables

GEI Consultants	Client	Town of Bar Harbor			Page	1
	Project	Bar Harbor Ferry Terminal			Pg. Rev.	
	By	DAP	Chk.	DJB	App.	
	Date	11/12/2020	Date	11/12/2020	Date	

Project No.	2004148	Document No.	--
Subject	Inspection Tables		

Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
P1-B	E	SV	SV	MJ	100% timber loss, 100% concrete loss, severe knifing @ low water
P1-Dnorth	D	SV	SV	MD	100% timber loss, 80% concrete loss, moderate steel corrosion, biting
P1-Dsouth	E	SV	SV	MJ	100% timber loss, 80% concrete loss, severe hour glassing to web, severe biting from low water to mudline/rubble
P1-Fnorth	D	SV	SV	MJ	100% timber loss, 80% concrete loss, knifing and biting @ low water, moderate steel corrosion through out
P1-Fsouth	E	SV	SV	SV	100% concrete loss. Severe hour glassing at low water, knifing at Elevation +1' to +4'.
P1-H	D	SV	SV	MJ	90% concrete loss, moderate steel corrosion, knifing at low water, biting at low water
P2-B	C	MD	MD	MD	Full timber cover, moderate steel corrosion
P2-C	C	MD	MD	MD	Full timber cover, moderate corrosion
P2-Dnorth	C	SV	SV	MD	100% concrete loss, moderate steel corrosion
P2-Dsouth	D	SV	SV	MJ	80% concrete loss, biting and knifing at low water
P2-Fnorth	D	SV	SV	MJ	50% timber loss below high water, 60% concrete loss, moderate steel corrosion through out, knifing at low water
P2-Fsouth	C	SV	SV	MD	40% concrete loss, moderate steel corrosion throughout
P2-G	D	SV	SV	MJ	30% concrete loss throughout, moderate knifing @ low water
P2-H	D	SV	SV	MJ	40% concrete loss, knifing at low water

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Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
P3-B	C	MD	NI	MD	Full timber, concrete unknown
P3-Cnorth	D	MJ	SV	MJ	30% concrete loss, knifing @ low
P3-Csouth	D	SV	SV	MJ	100% concrete loss, repaired, repair in good condition, flange bent beneath repair
P3-Dnorth	F	SV	SV	SV	Severe hour glassing down to web on the sides at low water
P3-Dsouth	D	SV	SV	SV	Severe biting and knifing at low water
P3-Fnorth	E	SV	SV	SV	70% concrete loss, biting to flange down to web
P3-Fsouth	C	SV	SV	MD	Repair in satisfactory condition, minor corrosion below low water
P3-Gnorth	D	SV	SV	MJ	30% timber loss, moderate steel corrosion, knifing at low
P3-Gsouth	D	SV	SV	MJ	50% timber loss, 80% concrete loss, moderate steel corrosion, knifing at low water
P3-H	C	MJ	SV	MD	Full timber cover, timber deterioration below MLW, concrete mostly off flanges moderate steel corrosion
P4-B	D	SV	SV	MJ	Severe timber loss below mean low water, 50% concrete loss, knifing and biting at low water
P4-C	D	SV	SV	MJ	Severe timber loss below mean low water, 80% concrete loss, knifing and biting (1"-2") at low water
P4-Dnorth	D	SV	SV	MJ	Severe timber loss below mean low water, knifing and biting at low water, knifing and 2" biting

Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
P4-Dsouth	C	SV	SV	MD	Pile repair in good condition
P4-Fnorth	E	SV	SV	MJ	20% concrete loss, knifing at low water
P4-Fsouth	F	SV	SV	SV	24" height Full timber loss below water, 80% concrete loss, Complete hour glassing down to web
P4-G	C	MD	NI	NI	Jacket to mud, concrete condition unknown
P4-H	D	SV	SV	MJ	Severe timber loss below MLW, concrete off flanges, moderate steel corrosion with moderate knifing
P5-B	E	SV	SV	SV	Biting down to web at low water, 4' in length. Slab surrounding pile is collapsed.
P5-Dnorth	F	SV	SV	SV	Complete hour glassing at low water, 20" in length. Slab surrounding pile is collapsed.
P5-Dsouth	E	SV	SV	SV	Severe knifing and biting on two flange ends at low water
P5-Fnorth	F	SV	SV	SV	90% concrete loss, hour glassing at low water
P5-Fsouth	D	SV	SV	MJ	30% concrete loss, moderate knifing, severe pitting
P5-H	D	SV	SV	MJ	Severe timber loss below mean low water, 40% concrete loss, moderate knifing, severe pitting
P5A-C	C	SV	SV	MD	100% timber loss, 30% concrete corrosion, slab surrounding pile is collapsed.
P5A-E	E	SV	SV	SV	Hour glassing near low water (18" in length), 10% concrete loss. Slab surrounding pile is collapsed.
P5A-G	C	MJ	NI	MD	Full timber jacket, moderate steel corrosion
P5B-C	C	MJ	NI	MD	Full timber, moderate steel corrosion

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Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
P5B-E	F	SV	SV	SV	100% timber loss, 90% concrete loss, hour glassing to web (3' in length)
P5B-G	D	SV	SV	MJ	80% concrete loss, heavy corrosion at low water,
P6-B	D	SV	MJ	MJ	Moderate knifing and biting at low water
P6-Dnorth	D	MJ	MJ	MD	20% concrete loss, moderate steel corrosion through out
P6-Dsouth	C	SV	MJ	MJ	Minor knifing at low water
P6-Fnorth	C	MJ	NI	MD	Full timber jacket, concrete unknown, moderate steel corrosion
P6-Fsouth	D	SV	SV	MJ	70% concrete loss, knifing at low water
P6-H	C	SV	SV	MD	Concrete 80%, moderate steel corrosion at low water
P7-B	E	SV	SV	MJ	No timber, concrete off of flanges. Severe steel corrosion, knifing and biting at low water.
P7-Dnorth	D	SV	MJ	MJ	Moderate to severe steel corrosion at low water, knifing at low water
P7-Dsouth	C	SV	SV	MD	No timber, concrete mostly off flanges, repair is in good condition, n
P7-Fnorth	D	MJ	MJ	MJ	Knifing, minor biting
P7-Fsouth	D	SV	SV	MD	70% concrete loss, moderate steel corrosion
P7-H	D	SV	SV	MD	100% timber loss, moderate steel corrosion
P8-B	D	SV	SV	MJ	80% loss of concrete, knifing at mean low.

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Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
P8-Dnorth	D	SV	SV	MJ	Severe timber loss below mean low, 80% concrete loss, start of knifing
P8-Dsouth	E	SV	SV	SV	Severe timber loss below mean low, 80% concrete loss, severe steel corrosion
P8-Fnorth	F	SV	SV	SV	Severe timber loss below mean low, 80% concrete loss, severe steel corrosion, hole in web (photo)
P8-Fsouth	D	SV	SV	MJ	Severe timber loss below mean low, 50% concrete loss, knifing and biting
P8-H	E	MJ	SV	SV	Timber loss below mean low, severe concrete loss underneath
P9-B	E	SV	SV	SV	Damage to timber below MLW, 50% concrete loss, severe steel corrosion, start of knifing at mean low
P9-Dnorth	E	SV	SV	SV	Severe timber loss below mean low water, 80% concrete loss, severe steel corrosion, start at biting at mean low
P9-Dsouth	F	SV	SV	SV	Severe timber loss below mean low water, 80% concrete loss, severe steel corrosion, knifing, biting (large on both flanges) at mean low
P9-Fnorth	E	SV	SV	SV	Severe timber loss and concrete below MLW, 90% concrete loss, hole in web, has channel repairs
P9-Fsouth	F	SV	SV	SV	Severe timber loss, severe concrete section loss, knecking (photos)
P9-H	E	SV	SV	SV	Severe timber loss below mean low water. Concrete mostly off flanges. moderate steel corrosion.
P10-B	E	SV	SV	SV	Severe timber loss, 60% concrete loss, severe steel corrosion
P10-Dnorth	E	SV	SV	SV	Severe timber loss below mean low water, 50% concrete loss, knifing at mean low
P10-Dsouth	E	SV	SV	SV	60% concrete loss, severe steel corrosion, knifing at mean low, severe timber loss below mean low water
P10-Fnorth	E	SV	SV	SV	Severe timber loss below mean low water, 40% concrete loss, severe steel corrosion, biting at mean low

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Subject	Inspection Tables		

Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
P10-Fsouth	E	SV	SV	SV	No timber below mean low water, 60% concrete loss, knifing at mean low,. Steel thickness measured with Ultrasonic Thickness Gauge.
P10-H	D	MJ	SV	MJ	40% concrete loss, concrete beginning to come apart at mean low water, severe steel corrosion
P11-B	C	MJ	MJ	MD	Concrete still between flanges, moderate steel corrosion, typical
P11-Cnorth	F	SV	SV	SV	Severe timber loss, 50% concrete loss, knifing throughout, biting at mean low on both sides of flanges
P11-Csouth	E	SV	SV	SV	70% concrete loss, knifing at mean low
P11-Dnorth	D	SV	SV	MD	Severe timber loss below MLW, 80% concrete loss, channels on outside (repaired) severe corrosion
P11-Dsouth	D	SV	SV	MJ	Severe timber loss below MLW, 70% concrete loss, severe steel corrosion, knifing at mean low
P11-Enorth	D	SV	SV	MJ	Severe timber loss below MLW, 80% concrete loss, moderate steel corrosion, knifing at mean low
P11-Esouth	D	SV	SV	MJ	Severe timber loss below MLW, 80% concrete loss, moderate steel corrosion, knifing at mean low
P11-Fnorth	D	SV	SV	MJ	No timber, 70% concrete loss, knifing at mean low
P11-Fsouth	D	SV	SV	MJ	Severe timber loss below MLW, 50% concrete loss, severe steel corrosion
P11-H	D	SV	SV	MJ	Timber has slid down 5 feet Timber has slid down 5 feet, 70% concrete loss, knifing at mean low
P12-B	D	SV	SV	MJ	50% concrete loss, knifing at mean low
P12-Cnorth	E	SV	SV	SV	80% concrete section loss, severe knifing and biting at mean low. Steel thickness measured with Ultrasonic Thickness Gauge.
P12-Csouth	F	SV	SV	SV	No timber, 90% concrete loss, split at weld

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Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
P12-Dnorth	D	SV	SV	MJ	80% concrete loss, knifing and biting at mean low
P12-Dsouth	D	SV	SV	MJ	80% concrete loss (photo), hole in web, two bolted channels
P12-Enorth	D	SV	SV	MJ	80% concrete loss, severe corrosion
P12-Esouth	E	SV	SV	SV	80% concrete loss, severe steel corrosion, knifing at mean low
P12-Fnorth	E	SV	SV	SV	40% concrete loss, severe knifing
P12-Fsouth	D	SV	SV	MJ	Severe timber loss below mean low water, 30% concrete loss, biting at mean low
P12-H	E	SV	SV	SV	30% concrete loss, knifing at mean low
P13-B	C	SV	SV	MD	50% concrete loss, moderate steel corrosion
P13-Dnorth	C	SV	SV	MJ	Severe timber loss, 60% concrete loss, severe corrosion at mean low
P13-Dsouth	E	SV	SV	SV	Severe timber loss, typical concrete loss, 70% concrete corrosion, severe biting at mean low,
P13-Fnorth	E	SV	SV	SV	Severe timber loss, 70% concrete corrosion, severe biting at mean low, typical concrete loss
P13-Fsouth	C	SV	SV	MJ	Severe timber loss, 60% concrete loss, severe corrosion at mean low, has repairs (channels)
P13-H	E	SV	SV	SV	80% concrete loss, severe knifing at mean low
P14-B	E	SV	SV	SV	50% concrete loss, severe steel corrosion at mean low
P14-Dnorth	E	SV	SV	SV	50% concrete loss, severe steel corrosion at mean low. Steel thickness measured with Ultrasonic Thickness Gauge.

Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
P14-Dsouth	D	SV	SV	MJ	Severe timber loss, 80% concrete loss, knifing at mean low (typical knifing), repaired
P14-Fnorth	F	SV	SV	SV	Severe timber loss,80% concrete loss, 1.5' hole (photo)
P14-Fsouth	F	SV	SV	SV	Severe timber loss,70% concrete loss, hole in web (photo)
P14-H	E	SV	SV	SV	100% timber loss, severe steel corrosion, 50% concrete loss
P15-B	C	SV	SV	MD	Severe timber loss, 50% concrete loss, moderate steel corrosion
P15-Dnorth	C	SV	SV	MD	Severe timber loss,50% concrete loss, moderate steel corrosion
P15-Dsouth	C	SV	SV	MD	60% concrete loss, moderate steel corrosion
P15-Fnorth	D	SV	SV	MJ	50% concrete loss, knifing at mean low
P15-Fsouth	D	SV	SV	MJ	Severe timber loss below MLW, 40% concrete loss
P15-H	E	SV	SV	SV	40% concrete loss, severe knifing and 2" biting at mean low
P15A-C	F	SV	SV	SV	No timber, pile necked to 1" (photo)
P15A-E	C	MJ	SV	MD	Severe timber loss below MLW, 50% concrete loss, moderate steel corrosion
P15A-G	C	MJ	SV	MD	50% concrete loss, moderate steel corrosion
P15B-C	C	MJ	SV	MD	Angles around perimeter near mean low, 60% concrete loss
P15B-E	E	SV	SV	SV	60% concrete section loss, severe knifing
P15B-G	D	SV	SV	MD	Severe timber loss, 50% concrete loss, moderate steel corrosion

Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
P16-B	E	SV	SV	SV	Bite through flange 3' long and 4" deep. Steel thickness measured with Ultrasonic Thickness Gauge.
P16-Dnorth	D	SV	SV	MJ	50% concrete loss, severe steel corrosion
P16-Dsouth	E	SV	SV	SV	60% concrete loss, biting at mean low, severe timber loss
P16-Fnorth	F	SV	SV	SV	50% concrete loss, severe knifing and biting to web, severe timber loss
P16-Fsouth	E	SV	SV	SV	60% concrete loss, severe knifing and biting at mean low, severe timber loss
P16-H	D	SV	SV	MJ	80% concrete loss, knifing at mean low, severe timber loss.
P17-B	D	SV	SV	MJ	80% concrete loss, knifing at mean low
P17-C	C	SV	SV	MD	Jacketed halfway down, 50% concrete loss, moderate steel corrosion
P17-Dnorth	D	SV	SV	MJ	50% concrete loss, severe corrosion
P17-Dsouth	F	SV	SV	SV	Severe hour glassing, photo
P17-Fnorth	C	SV	SV	MD	Has channel, 90% concrete loss (repaired)
P17-Fsouth	C	SV	SV	MD	Channels at base, 100% timber loss, severe corrosion (repaired)
P17-G	E	SV	SV	SV	80% concrete loss, severe steel corrosion
P17-H	E	SV	SV	SV	80% concrete loss, knifing and biting at mean low, severe timber loss below MLW
P18-B	E	SV	SV	SV	80% concrete loss, severe steel corrosion, knifing at mean low

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Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
P18-Cnorth	E	SV	SV	SV	90% concrete loss, severe knifing, 4" bites (1.5' long) on flange near mean low
P18-Csouth	D	SV	SV	MJ	30% concrete loss, moderate steel corrosion, knifing beginning to show at mean low
P18-Dnorth	D	SV	SV	MD	40% concrete loss, moderate steel corrosion throughout
P18-Dsouth	D	SV	SV	MJ	40% concrete loss, knifing through intertidal, severe timber loss
P18-Fnorth	D	SV	SV	MJ	40% concrete loss, knifing at mean low, severe timber loss
P18-Fsouth	E	SV	SV	SV	50% concrete loss, knifing through out, biting to web on both flanges at mean low, severe timber loss
P18-Gnorth	E	SV	SV	SV	50% concrete loss, knifing through out, biting to web on both flanges at mean low, severe timber loss. Steel thickness measured with Ultrasonic Thickness Gauge.
P18-Gsouth	D	SV	SV	MJ	100% concrete loss, severe steel corrosion, steel flaking, severe timber loss
P18-H	C	SV	SV	MD	30% concrete loss, moderate steel corrosion, no timber below MLW, concrete off flanges
P19-B	C	SV	SV	MD	No timber, 40% concrete loss, moderate steel corrosion
P19-C	C	SV	SV	MD	20% concrete loss, moderate steel corrosion, severe timber loss
P19-Dnorth	C	SV	SV	MD	30% concrete loss, moderate steel corrosion, severe timber loss
P19-Dsouth	C	SV	SV	MD	40% concrete loss, severe timber loss
P19-Fnorth	E	SV	SV	MJ	50% concrete loss, knifing at mean low, severe timber loss
P19-Fsouth	C	SV	SV	MD	100% timber loss, 50 % concrete loss, moderate steel corrosion & pitting
P19-G	D	SV	SV	MJ	90% concrete loss, no timber, knifing, biting to 1" from web (photo), severe timber loss

Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
P19-H	D	SV	MJ	MJ	40% concrete loss, severe corrosion, evidence of knifing
P20-B	C	SV	SV	MD	80% concrete loss, minor corrosion
P20-Dnorth	C	MJ	MJ	MD	10% concrete loss, moderate steel corrosion
P20-Dsouth	D	SV	SV	MJ	40% concrete loss, no timber. Steel thickness measured with Ultrasonic Thickness Gauge.
P20-Fnorth	D	SV	SV	MJ	70% concrete loss, knifing through out, 50% timber, severe timber loss
P20-Fsouth	D	SV	SV	MJ	30% concrete loss, moderate steel corrosion, minor knifing, severe timber loss
P20-H	C	SV	SV	MD	30% concrete loss, moderate steel corrosion, 50% timber, severe timber loss
1-V	NI	NI	NI	NI	Pile Buried in rip rap
1-W	NI	NI	NI	NI	Pile Buried in rip rap
1-Y	NI	NI	NI	NI	Pile Buried in rip rap
1-Z	NI	NI	NI	NI	Pile Buried in rip rap
2-V	NI	MD	NI	NI	Moderate rot and section loss in timber. Pile not inspected due to exposed portion being fully embedded below grade.
2-W	NI	MD	NI	NI	Moderate rot and section loss in timber. Pile not inspected due to exposed portion being fully embedded below grade.
2-Y	NI	MD	NI	NI	Moderate rot and section loss in timber. Pile not inspected due to exposed portion being fully embedded below grade.
2-Z	NI	MD	NI	NI	Moderate rot and section loss in timber. Pile not inspected due to exposed portion being fully embedded below grade.
3-V	D	SV	MJ	MJ	Steel moderate to severe, flanges knifing, 80% loss of concrete, steel exposed. Timber splintering.
3-W	C	SV	MN	MD	Timber severely deteriorated on all piles. Concrete minor corrosion.
3-Y	C	SV	MN	MD	Timber severely deteriorated on all piles. Concrete minor corrosion.
3-Z	C	SV	MN	MD	Timber severely deteriorated on all piles. Concrete minor corrosion.
4-V	D	SV	SV	MJ	Timber severe, severe loss of concrete, moderate steel section loss/knifing. Steel flakes (typical).

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Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
4-W	C	SV	MD	MD	Timber severe, moderate concrete loss. Moderate steel corrosion.
4-Y	C	SV	MD	MD	Minor knifing and pitting, with flaking. Moderate concrete loss. Damage to timber.
4-Z	D	MJ	MJ	MJ	Minor knifing, minor pitting. Timber minor deteriorated. Steel thickness measured with Ultrasonic Thickness Gauge.
5-V	D	SV	SV	MJ	Knifing steel, moderate to major biting, 50% concrete loss, timber severe (typ. Major loss).
5-W	E	SV	SV	MJ	Timber in severe condition, 100% concrete loss from elevation -2 to +4. moderate steel corrosion with biting.
5-Y	D	SV	SV	MJ	100% concrete loss below water, steel moderate (biting), Timber severe (type). 100% timber loss. Steel thickness measured with Ultrasonic Thickness Gauge.
5-Z	D	SV	SV	MJ	Approx. 50% concrete loss below water, steel moderate (biting), Timber severe (type). 100% timber loss.
6-V	D	SV	MJ	MJ	Moderate corrosion of steel, little knifing/pitting, <20% concrete loss below water, timber severe (typ.)
6-W	C	MD	MD	MD	Timber poor, timber debris on bottom, <15% concrete loss
6-Y	C	MD	MD	MD	Timber poor, timber debris on bottom, <15% concrete loss
6-Z	D	MD	MJ	MJ	Timber fully missing, concrete between flanges only to MLW, <15% concrete loss. Steel thickness measured with Ultrasonic Thickness Gauge.
7-V	C	MD	MD	MD	Moderate concrete loss, moderate steel corrosion, timber intact. Damage to timber.
7-W	F	SV	SV	SV	Severe knifing, necking down 3" each flange timber severe. Steel thickness measured with Ultrasonic Thickness Gauge.
7-Y	C	SV	SV	MD	50% concrete loss, moderate steel corrosion
7-Z	C	SV	MJ	MD	Damage to timber, >30% concrete corrosion, moderate steel corrosion
8-Vnorth	D	SV	MJ	MJ	Portion of timber gone, 70% concrete intact, moderate steel corrosion
8-Vsouth	C	MJ	NI	MD	Timber intact - moderate/severe. Moderate steel corrosion.

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Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
8-Y	E	MJ	MD	SV	Timber intact - moderate/severe. Severe steel corrosion. Steel thickness measured with Ultrasonic Thickness Gauge.
8-Z	C	MJ	MD	MD	Timber intact - moderate/severe. Moderate steel corrosion.
9-R	C	SV	SV	MD	Severe concrete loss, severe timber rot, moderate steel corrosion. 25% timber loss. Moderate/severe concrete loss, 4-5' section is gone.
9-S	C	SV	MJ	MD	Moderate to severe concrete loss. 40% timber loss. 4-5' section is gone.
9-T	C	SV	MJ	MD	Typical concrete loss, moderate steel corrosion, 50% timber
9-U	C	MJ	MJ	MD	Jacket intact, moderate steel corrosion, moderate concrete loss
9-V	C	SV	NI	MD	moderate steel corrosion, moderate concrete corrosion
9-X	C	SV	MJ	MD	moderate steel corrosion, moderate concrete corrosion, 25% timber loss
9-Z	C	MJ	MJ	MD	moderate steel corrosion, moderate concrete corrosion
10-R	D	SV	SV	MD	moderate steel corrosion, 30% concrete loss, 75% timber loss below MLW
10-S	E	SV	SV	MJ	100% concrete loss below MLW, knifing 3", timber's gone below low water
10-T	C	SV	SV	MD	moderate steel corrosion, 30% concrete loss, timber 100% loss starting 6' below slab
10-U	C	MJ	MJ	MD	30% concrete loss, moderate steel corrosion
10-V	E	SV	SV	SV	100% timber loss and concrete loss from mid to low water, knifing through elevation and biting
10-X	C	SV	SV	MD	50% timber loss. Severe concrete loss below MLW. Moderate corrosion of steel below MLW.
10-Z	D	SV	SV	MJ	100% timber loss below high water, moderate to severe concrete loss in intertidal zone, moderate to severe steel
11-R	C	SV	SV	MD	Timber typical (50% loss), moderate steel corrosion. 50% concrete loss

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Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
11-S	C	SV	SV	MD	40% timber loss. Moderate to severe concrete loss above MLW. Moderate steel corrosion below MLW.
11-T	D	SV	SV	MJ	Concrete loss to web, 50% timber Moderate to severe concrete loss above MLW. Steel moderate corrosion below MLW. Steel thickness measured with Ultrasonic Thickness Gauge.
11-U	F	SV	SV	SV	Bite all the way to web, no timber. Concrete between flanges.
11-V	E	SV	SV	SV	Concrete in web remains, 80% concrete loss otherwise. Concrete spall at web. Severe steel corrosion. No timber.
11-X	D	SV	SV	MD	Loose timber through out height, moderate steel corrosion, moderate concrete. Concrete spalling above MLW, moderate steel corrosion below MLW.
11-Z	D	SV	SV	MD	Loose timber, 30% concrete loss, moderate steel corrosion
12-R	D	SV	SV	MJ	Complete timber loss below high water, no concrete on flanges, spalled concrete on web, knifing in intertidal
12-S	C	SV	SV	MD	Loose timber, moderate concrete loss, minor corrosion, typical steel corrosion
12-T	F	SV	SV	SV	Heavy biting down to web at low water, partial concrete and deterioration (50%), no timber below high water
12-U	E	SV	SV	SV	Biting, mean low water to +4 heavy biting and knifing, severe steel corrosion beneath concrete, timber exposed, timber splintering
12-V	D	SV	SV	MD	Timber several corroded and concrete off flanges below MLW, moderate steel corrosion
12-X	D	SV	SV	MD	Moderate steel corrosion, spalled concrete (light), timber splintering (50%)
12-Z	C	SV	SV	MD	No timber past highwater, no concrete past high water, steel corrosion minor, minor knifing from mid height to low water
13-R	F	SV	SV	SV	Knifing and biting to web, 6" hole in web near El. +2, no timber below MLW, concrete off flanges below MLW
13-S	D	SV	SV	MJ	Minor knifing and biting throughout intertidal, timber coming off, severe timber loss below MLW
13-T	F	SV	SV	SV	100% timber loss, 50% concrete loss from high to low, severe steel corrosion throughout. Biting 24" to web @ mean low water. Steel thickness measured with Ultrasonic Thickness Gauge.
13-U	F	SV	SV	SV	Necking on steel, knifing from high to low, severe biting to web on all sides. No timber below MLW.

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Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
13-V	E	SV	SV	SV	Minor biting from mean low to +4', biting at mean low. No concrete or timber near mean low water
13-X	E	SV	SV	SV	No timber, no concrete below low water, biting at low water, knifing too
13-Z	D	SV	SV	MJ	No timber below high water, mid intertidal to mean low concrete loss, at mean low biting
14-R	C	SV	SV	MD	Timber poor, 20% concrete loss at web, 100% concrete loss on flanges, moderate pitting
14-S	E	SV	SV	SV	50% concrete loss in web, no concrete on flange, 3-4' knifing after concrete ends, severe pitting
14-T	E	SV	SV	SV	Biting all flanges down to web, poor timber, concrete missing from end of timber to low water, severe pitting
14-U	D	SV	SV	MJ	Very poor timber ends at high water, 60% concrete between flanges, knifing 3' below mean low
14-V	C	SV	SV	MD	Timber's poor (75% loss below MLW), 30% concrete loss, typical pitting, concrete mostly off flanges below MLW
14-X	C	SV	SV	MD	Timber to mean low, concrete typical loss, moderate steel corrosion
14-Z	C	MD	NI	MD	Moderate steel corrosion, concrete not visible through timber
15-R	E	SV	SV	SV	80-90% concrete loss, knifing, severe steel corrosion
15-S	E	SV	SV	SV	80-90% concrete loss, severe knifing, biting from mean low to -2'. Bite (1') near mean low. severe timber loss. Steel thickness measured with Ultrasonic Thickness Gauge.
15-T	D	SV	SV	MJ	Concrete missing to mean low water, knifing through out, severe timber loss below MLW
15-U	D	SV	SV	MJ	No timber. 50% concrete missing, minor knifing
15-V	D	SV	SV	MJ	50-70% concrete loss, poor timber, moderate corrosion on steel, timber severely corroded below MLW,
15-X	C	SV	MJ	MD	Timber poor, moderate steel corrosion, 30% concrete loss
15-Z	D	SV	SV	MJ	80% concrete loss, knifing through out, biting near mean low

Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
16-R	E	SV	SV	SV	Severe timber degradation. Knifing to mean low. Biting at mean low, concrete 90% gone.
16-S	E	SV	SV	SV	90% concrete loss, knifing to mean low, biting at mean low, no timber below mean low.
16-T	E	SV	SV	SV	90% concrete loss, moderate knifing, hour glassing at mean low, no timber below mean low
16-U	D	SV	SV	MJ	90% concrete loss, severe steel corrosion. No timber.
16-V	D	SV	SV	MJ	90% concrete loss, severe steel corrosion through out
16-X	D	SV	SV	MD	40% concrete loss, moderate steel corrosion, 50% timber
16-Z	C	SV	MJ	MD	10% concrete loss, moderate steel corrosion, 50% timber
17-V	C	SV	SV	MD	30% concrete loss, moderate steel corrosion, concrete off flanges, severe timber loss below MLW
17-W	E	SV	SV	SV	90% concrete loss, severe steel corrosion, knifing at mean low
17-Y	E	SV	SV	SV	100% concrete loss, severe steel corrosion, knifing at mean low
17-Z	C	SV	SV	MD	20% concrete loss, moderate steel corrosion, no timber
18-V	C	SV	SV	MD	20% concrete loss, moderate steel corrosion, no timber below mean low, concrete off flanges below MLW
18-W	C	MJ	SV	MD	10% concrete loss, moderate steel corrosion
18-Y	C	MJ	MJ	MD	20% concrete loss, moderate steel corrosion. Steel thickness measured with Ultrasonic Thickness Gauge.
18-Z	C	SV	SV	MD	30% concrete loss, moderate steel corrosion
19-V	D	SV	SV	MJ	90% concrete loss, severe corrosion, severe timber loss below mean low
19-W	F	SV	SV	SV	100% concrete loss, hour glassing at mean low, 5"x10" hole, 100% concrete loss

Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
19-Y	D	SV	SV	MD	50% concrete loss, moderate steel corrosion, no timber below mean low
19-Z	D	SV	SV	MD	80% concrete loss, moderate steel corrosion
20-V	E	SV	SV	MJ	90% concrete loss, severe corrosion, knifing and biting at mean low. No timber. Steel thickness measured with Ultrasonic Thickness Gauge.
20-W	E	SV	SV	MJ	80% concrete loss, knifing at mean low, no timber below Mean Low
20-Y	E	SV	SV	MJ	80% concrete loss, knifing at mean low, no concrete below mean low
20-Z	E	SV	SV	MJ	90% concrete loss, knifing through out, bite at mean low (10"), no timber
21-V	C	MD	NI	MD	Full timber jacket, moderate steel corrosion
21-W	C	MJ	SV	MJ	30% concrete loss, starting to show evidence of knifing, moderate steel corrosion
21-Y	C	SV	SV	MJ	40% concrete loss, moderate steel corrosion, start knifing
21-Z	D	SV	SV	MJ	40% concrete loss, knifing at mean low, no timber
22-V	C	SV	SV	MD	Timber slide down pile 15', timber to mud. Concrete off flanges.
22-W	F	SV	SV	SV	90% concrete loss, hour glass to web @ mean low, knifing through out, 100%timber loss
22-Y	E	SV	SV	MJ	80% concrete loss, knifing from El. 0 to +3'
22-Z	C	SV	SV	MD	60% concrete loss, moderate steel corrosion
23-V	C	SV	SV	MD	40% concrete loss, moderate steel corrosion, severe timber deterioration below MLW, conc off flanges below mean low
23-W	C	SV	SV	MD	50% concrete corrosion, moderate corrosion steel, no timber
23-Y	D	SV	SV	MJ	60% concrete loss, knifing at mean low, no timber

Pile No.	Condition Rating (Table 2)	Damage Rating (Table 3)			Supplemental Notes
		Timber	Concrete	Steel	
23-Z	C	SV	SV	MJ	No timber, 50% concrete loss, knifing at mean low
24-W	C	SV	SV	SV	100% concrete loss, hourglass at mean low, hole in web, biting on all 4 flanges
24-Xeast	E	SV	SV	MJ	60% concrete loss, severe corrosion at mean low.
24-Xwest	C	SV	SV	MD	60% concrete loss, start of knifing. Steel thickness measured with Ultrasonic Thickness Gauge.
24-Y	C	SV	SV	MD	60% concrete loss, start of knifing
25-West	D	SV	SV	MJ	No timber, moderate steel corrosion, moderate evidence of knifing at low water, conc between flanges
25-Center W	D	SV	SV	MJ	No timber, moderate steel corrosion, moderate evidence of knifing at low water, conc between flanges
25-Center E	D	SV	SV	MJ	20% timber loss, moderate steel corrosion, moderate evidence of knifing at low water
25-East	D	SV	SV	MJ	No timber, moderate steel corrosion, concrete between flanges, moderate evidence of knifing at low water
26-West	C	SV	SV	MD	No timber, moderate steel corrosion
26-East	E	SV	SV	SV	Severe timber loss below mean low water, concrete off flanges below MLW, moderate to severe steel corrosion, severe biting and knifing on cross bracing
27-West	C	SV	SV	MD	No timber, concrete off flanges, moderate steel corrosion
27-East	F	SV	SV	SV	No timber, severe steel corrosion, biting to web @ low water (2' bite), hole in web 7" diameter at low water

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Subject	Ultrasonic Thickness Measurements		

Pile	Low Water Web Reading (in.)	Low Water Flange Reading (in.)	Mud Line Web Reading (in.)	Mud Line Flange Reading (in.)	High Water Web Reading (in.)	High Water Flange Reading (in.)	Electrical Potential (mV)
4-Z	n/a	0.525	n/a	0.275	n/a	n/a	225
5-Y	0.495	0.575	0.480	0.430	n/a	n/a	440 ave 480 max
6-Z	0.550	0.550	0.410	0.480	n/a	n/a	
7-W	0.585	0.220	0.470	0.430	n/a	n/a	375
8-Y	0.370	0.380	0.270	0.230	n/a	n/a	
11-T	n/a	0.580	0.380	0.375	n/a	n/a	315 @ mudline 330 @ low water
13-T	0.305	0.220	0.430	0.360	n/a	n/a	
15-S	0.285	0.220	0.455	0.460	n/a	n/a	
18-Y	0.535	0.535	0.500	0.500	n/a	n/a	450 @ mudline 350 @ low water
20-V	0.445	0.325	0.490	0.485	n/a	n/a	
24-Xwest	0.470	0.485	0.360	0.365	n/a	n/a	
P20-Dsouth	n/a	n/a	0.400	0.430	0.585	0.588	
P18-Gnorth	0.525	0.475	n/a	n/a	0.480	0.380	
P16-B	n/a	0.465	0.4600	0.4850	n/a	n/a	
P14-Dnorth	0.545	0.390	0.405	0.405	n/a	n/a	
P12-Cnorth	0.380	0.235	0.430	0.455	n/a	n/a	
P10-Fsouth	0.435	0.430	0.400	0.550	n/a	n/a	
P-8	0.420	0.400	0.440	0.325	n/a	n/a	

Appendix D

Inspection Photo Log

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Photo No. 1: Low tide view of facility from the west. Taken by DAP on 10/19/2020 at 2:52PM.

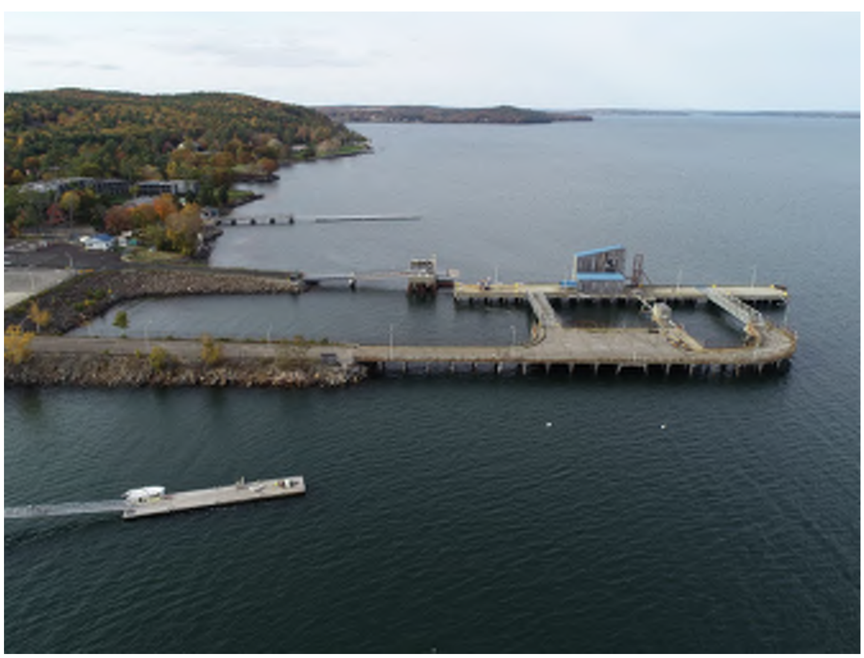

Photo No. 2: Low tide view of facility from the south. Taken by STH on 10/19/2020 at 2:57PM.

Photo No. 3: View of North Pier from the east. Taken by STH on 10/19/2020 at 3:10PM.

Photo No. 4: View of South Pier from the east. Taken by STH on 10/19/2020 at 3:11 PM.

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Photo No. 5: View of North Pier from above. Taken by STH on 10/19/2020 at 3:19PM.

Photo No. 6: View of East Vehicle Bridge. Taken by STH on 10/19/2020 at 3:27PM.

Photo No. 7: Typical topside conditions of North Pier. Taken by DAP on 10/20/2020 at 4:36PM.

Photo No. 8: Timber curb on northeastern corner of North Pier in poor condition. Bollards are in good condition. Taken by DAP on 10/20/2020 at 4:37PM.

Photo No. 9: Burned timber fender piles on eastern end of North Pier. Taken by STH on 10/19/2020 at 2:47PM.

Photo No. 10: Hazardous hanging guardrail near timber curb in poor condition. Taken by STH on 10/19/2020 at 2:48PM.

Photo No. 11: Typical condition of South Pier asphalt. Taken by DAP on 10/20/2020 at 4:26PM.

Photo No. 12 Typical sunken drain on South Pier. Taken by DAP on 10/24/2020 at 4:19PM.

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Photo No. 13: Hazardous downed fencing on southern side of revetment. Taken by DAP on 10/23/2020 at 9:19AM.

Photo No. 14: Downed fencing on South Pier. Taken by DAP on 10/23/2020 at 9:12AM.

Photo No. 15: Hole in asphalt on northern side of revetment. Taken by DAP on 10/23/2020 at 9:18AM.

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Photo No. 16: Topside conditions of West Vehicle Bridge. Taken by DAP on 10/23/2020 at 9:13AM.

Photo No. 17: View of catwalk from North Pier. Taken by DAP on 10/20/2020 at 4:32PM.

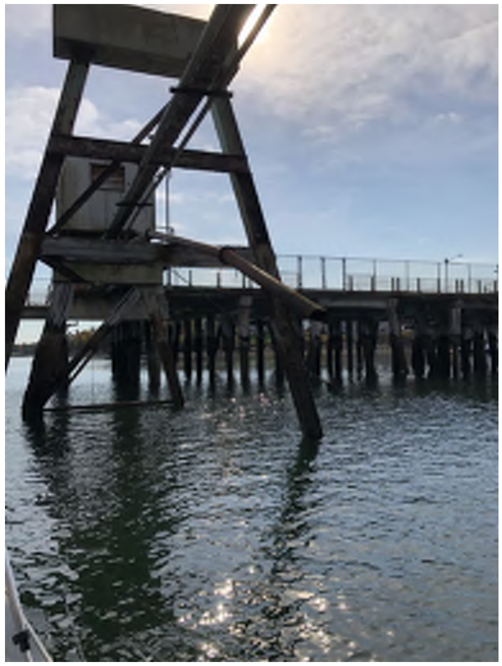

Photo No. 18: Steel pipe hanging from Pedestrian Bridge. Taken by DAP on 10/22/2020 at 11:12AM.

Photo No. 19: Corrosion on Pedestrian Bridge steel support structure. Taken by DAP on 10/23/2020 at 9:13AM.

Photo No. 20: Typical North Pier underside conditions. Taken by DAP on 10/22/2020 at 2:17PM.

Photo No. 21: Large spall on underside of eastern North Pier. Taken by DAP on 10/21/2020 at 3:48 PM.

Photo No. 22: Large spall on underside of the North Pier. Taken by DAP on 10/21/2020 at 3:46PM.

Photo No. 23: Severe spall on underside of North Pier. Taken by DAP on 10/22/2020 at 2:31PM.

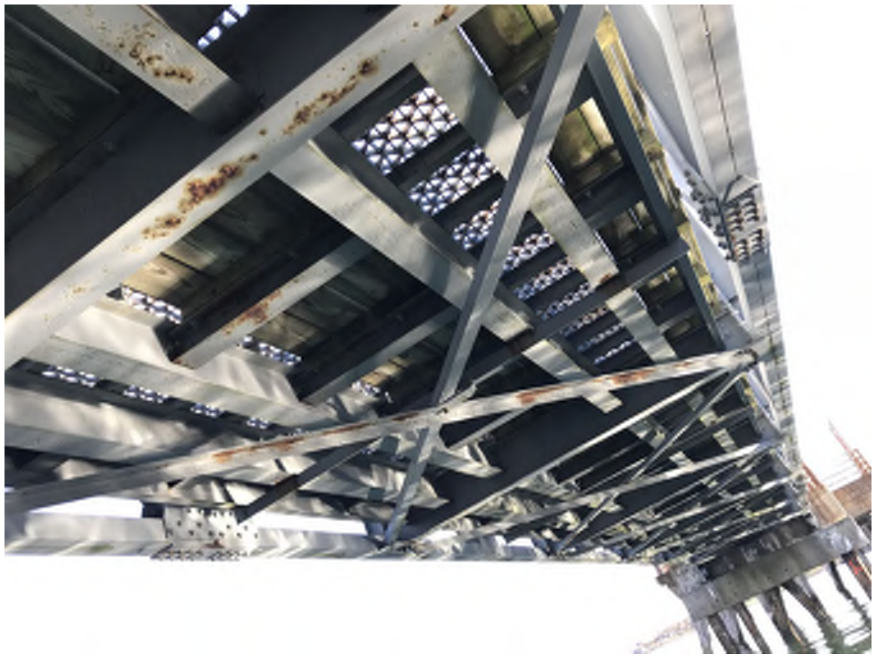

Photo No. 24: Typical Vehicle Ramp underside conditions. Taken by DAP on 10/22/2020 at 2:36PM.

Photo No. 25: Typical South Pier underside conditions. Taken by DAP on 10/22/2020 at 11:02AM.

Photo No. 26: Spalling on pile cap along bent 2. Taken by DAP on 10/23/2020 at 9:22AM.

Photo No. 27: Spalling on between bents 1 &3. Taken by DAP on 10/21/2020 at 3:27PM.

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Photo No. 28: Typical concrete corrosion exposing wire mesh. Taken by ASC on 10/21/2020 at 12:09PM.

Photo No. 29: Typical knifing on cross-brace structure. Taken by ASC on 10/22/2020 at 10:27PM.

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Photo No. 30: Typical condition of pile repairs. Taken by ASC on 10/22/2020 at 9:34AM.

Photo No. 31: Typical condition of pile repairs. Taken by ASC on 10/22/2020 at 9:34AM.

Photo No. 32: Severe efflorescence on underside of South Pier. Taken by DAP on 10/21/2020 at 3:11PM.

Photo No. 33: Typical sliding of timber jacketing. Taken by DAP on 10/22/2020 at 11:07PM.

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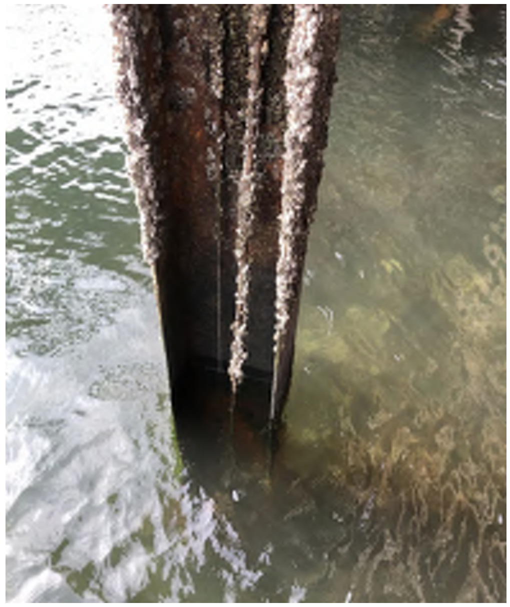


Photo No. 34: Steel H-pile with no remaining timber or concrete jacketing. Taken by ASC on 10/22/2020 at 11:10AM.

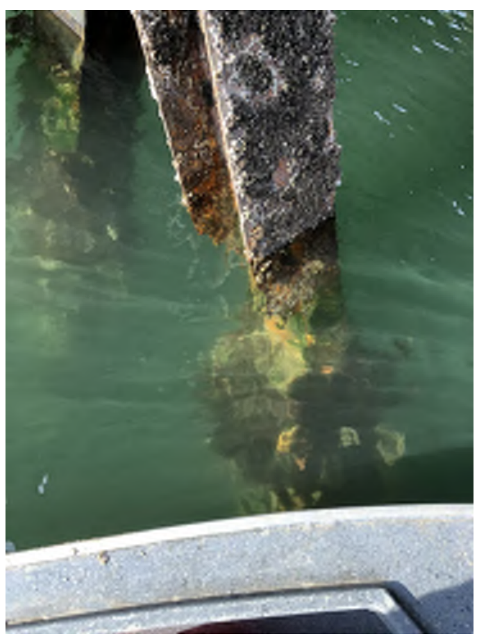


Photo No. 35: Typical knifing at mean low water. Taken by ASC on 10/22/2020 at 11:09AM.

GEI Consultants	Client	Town of Bar Harbor			Page	19/21
	Project	Ferry Terminal Inspection			Pg. Rev.	
	By	DAP	Chk.	DJB	App.	
	Date	10/26/2020	Date	11/5/2020	Date	
Project No.	2004148	Document No.	--			
Subject	Inspection Photo Log					

Photo No. 36: Typical pile with concrete corroded from flanges. Taken by ASC on 10/22/2020 at 11:05AM.

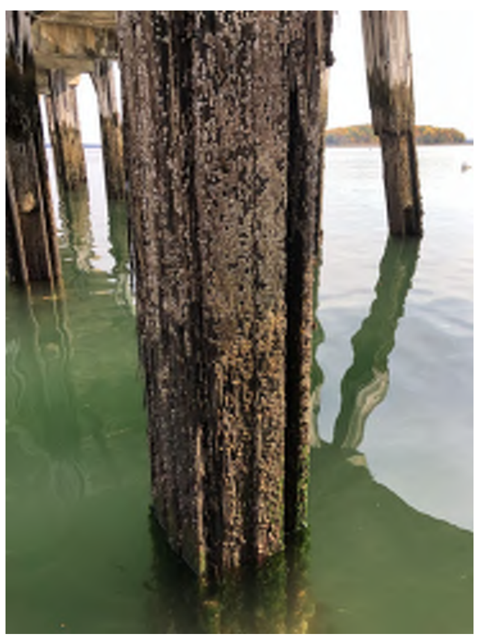


Photo No. 37: Typical condition of timber jacketing. Taken by ASC on 10/22/2020 at 11:00AM.

Photo No. 38: Typical corrosion of steel H-piles. Taken by ASC on 10/22/2020 at 1:04PM.

Photo No. 39: Hole in web of pile P14-Fsouth. Taken by ASC on 10/22/2020 at 1:21PM.

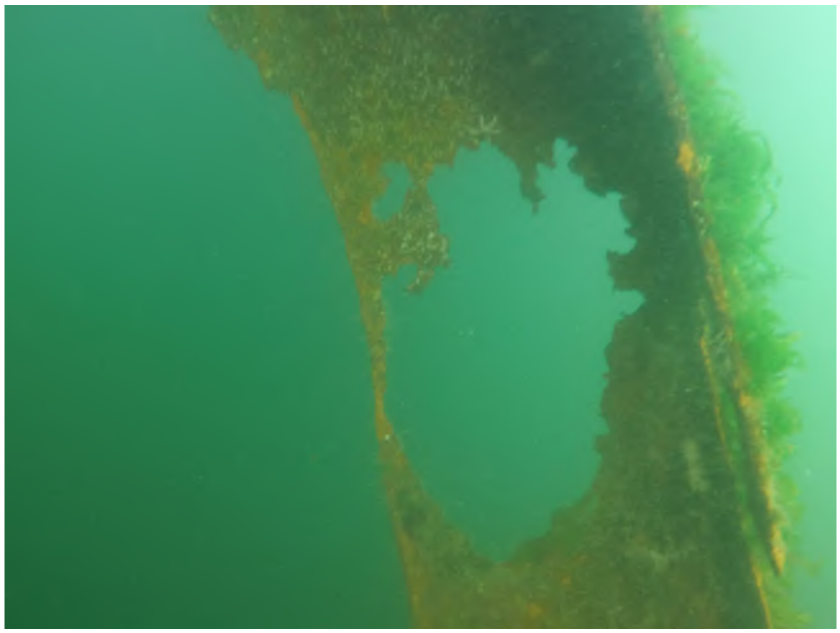
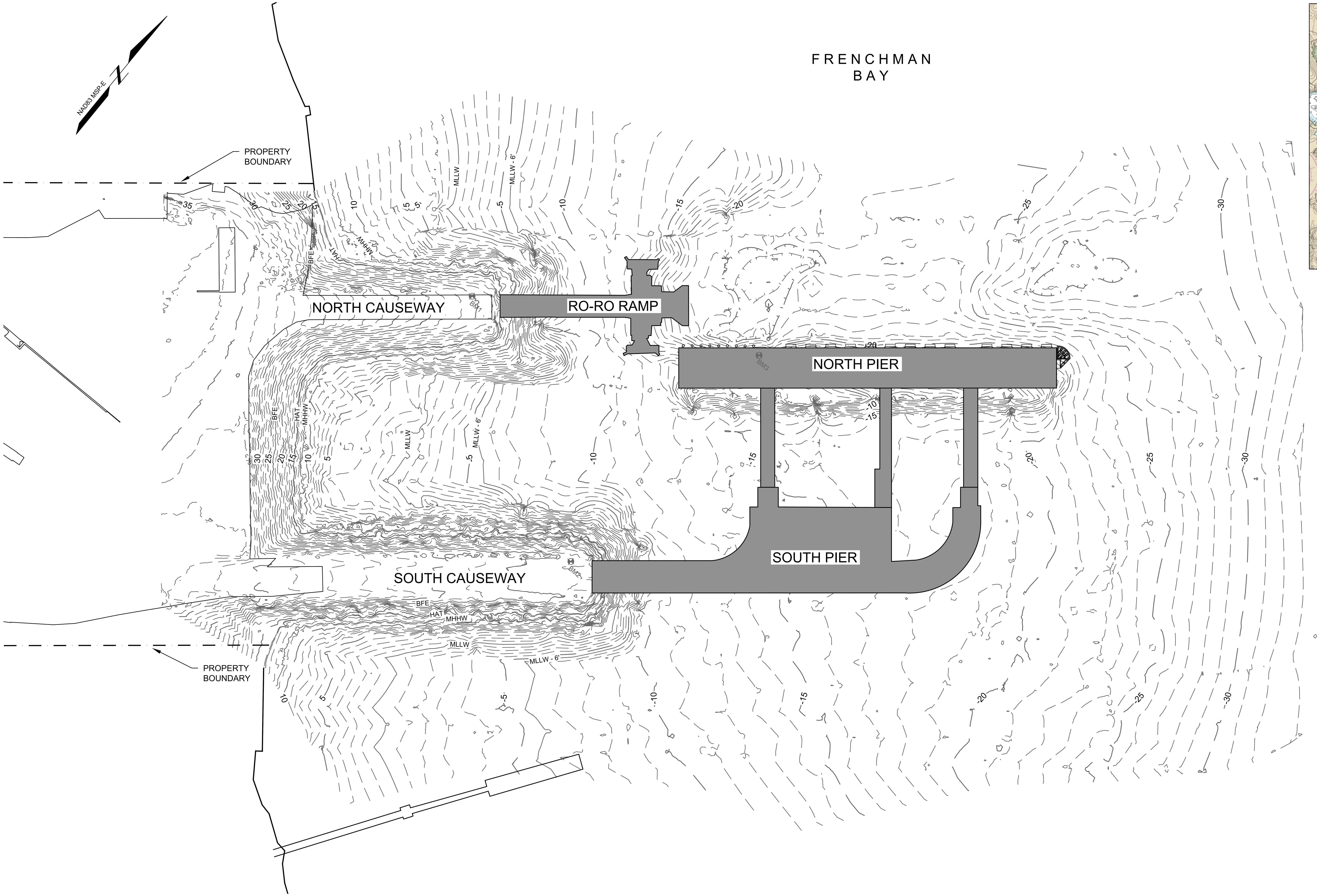


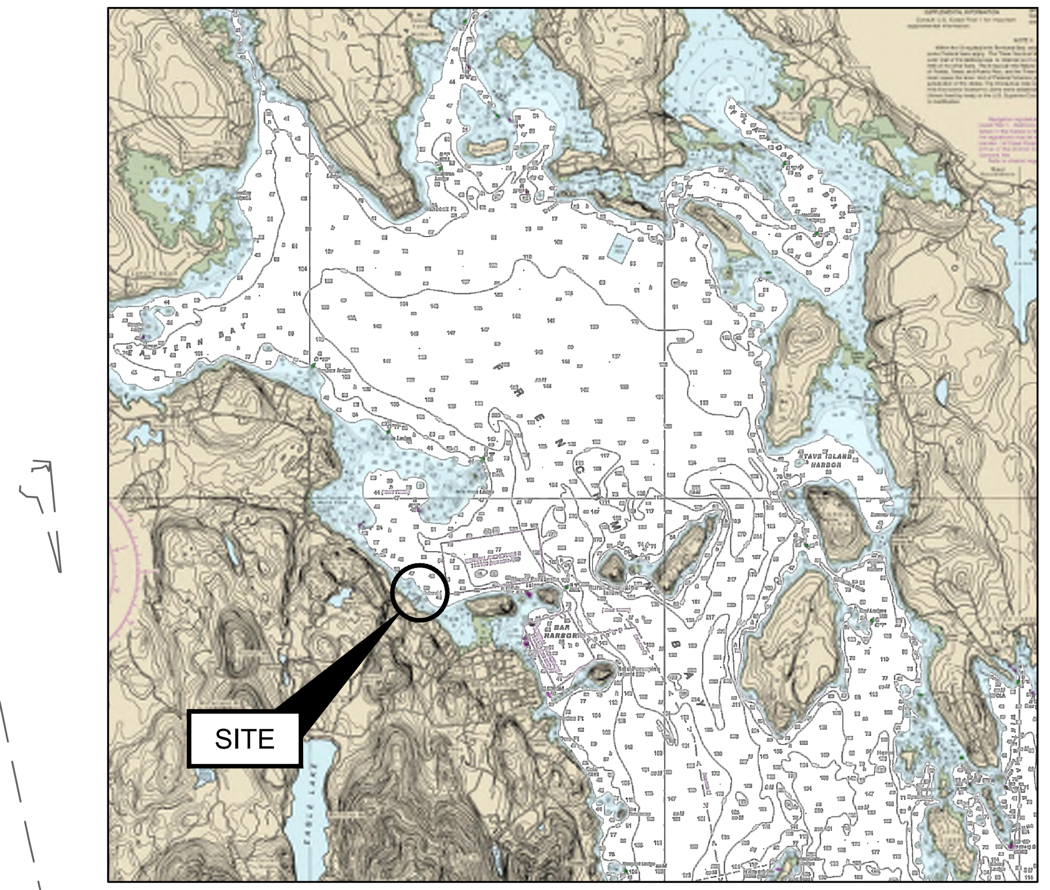
Photo No. 40: Hole in web of pile P14-Fnorth. Taken by ASC on 10/22/2020 at 1:30PM.

.

Appendix E

Hydrographic Survey Plan





NOAA CHART 13318 INSET

SCALE: 1:10,000

SURVEY AND PROPERTY NOTES:

1. LOCUS PARCEL IS DEPICTED ON TOWN OF BAR HARBOR TAX MAP 231, LOT 004 AND IS LOCATED IN THE S-5 SHORELAND MARITIME ACTIVITIES ZONE.
2. THE RECORD OWNER IS THE TOWN OF BAR HARBOR, BY DEED DATED JANUARY 23, 2019, RECORDED IN THE HANCOCK COUNTY REGISTRY OF DEEDS IN BOOK 6935, PAGE 340.
3. PORTIONS OF THE PROPERTY ARE DEPICTED AS BEING IN A SPECIAL FLOOD HAZARD AREA BASED ON FEMA FIRM 23009C1013D, EFFECTIVE DATE JULY 20, 2016. THE SITE IS LOCATED IN A VE ZONE HAVING BASE FLOOD ELEVATION OF 17 FT NAVD88, WHICH IS CONVERTED TO +22.97 FT MLLW.
4. BOUNDARY INFORMATION DEPICTED HEREON IS BASED ON A PLAN ENTITLED "BOUNDARY SURVEY PLAN OF PROPERTY OF STATE OF MAINE - MDOT" BY CES, INC. DATED JANUARY 22, 2019. UPLAND STRUCTURES AND FEATURES ARE DEPICTED BASED ON THAT SAME PLAN AND MAY NOT REFLECT CURRENT EXTENT OF CONSTRUCTION AT THE SITE.
5. TOPOGRAPHIC SURVEY DEPICTED HEREON WAS COLLECTED BY GEI CONSULTANTS, INC. ON OCTOBER 19, 2020 USING A DJI PHANTOM 4 DRONE WITH PHOTOGRAMMETRIC DATA COLLECTION AND POSTPROCESSED USING DRONEDEPLOY SOFTWARE.
6. HYDROGRAPHIC DATA DEPICTED HEREON WAS COLLECTED BY GEI CONSULTANTS, INC. ON OCTOBER 19, 2020 USING THE FOLLOWING EQUIPMENT:
   - TRIMBLE GEO XH 6000 SERIES GPS W/ ZEPHYR EXTERNAL WITH ANTENNA
   - ODEM ECHOTRAC MKII
   - SINGLE BEAM TRANSDUCER - 9° ANGLE - 200 kHz
   - HYPACK MAX HYDROGRAPHIC SURVEY SOFTWARE VERSION 2018 ON LAPTOP COMPUTER.
7. SITE CONTROL FOR HYDROGRAPHIC AND PHOTOGRAMMETRIC SURVEY WAS BASED ON THREE BENCHMARKS SET IN OCTOBER 2020 BY CES, INC.

BENCHMARK	NORTHING	EASTING	ELEVATION
BM1	267203.28	1055876.5	+19.97
BM2	267059.98	1056122.6	+22.36
BM3	267340.36	1056137.48	+21.85

8. HORIZONTAL COORDINATES ARE BASED ON NAD83 MAINE STATE PLANE EAST ZONE (1801) AND ARE EXPRESSED IN FEET.
9. ELEVATIONS ARE SHOWN IN FEET BASED ON MEAN LOWER LOW WATER (MLLW) DATUM. POSITIVE VALUES REPRESENT ELEVATION ABOVE THAT SAME PLANE. REFER TO ELEVATIONS TABLE FOR DATUM CONVERSIONS.
10. LIMITS OF EXISTING PIER AND RO-RO RAMP HAVE BEEN DIGITIZED BASED ON AERIAL PHOTOS AND HISTORIC PLANS AND SHOULD BE CONSIDERED APPROXIMATE.

PRELIMINARY

ELEVATION	MLLW*	NAVD88
BFE	+22.97	+17.00
HAT	+13.31	+7.34
MHHW	+11.37	+5.40
MHW	+10.94	+4.97
MSL	+5.67	-0.30
MLW	+0.38	-5.59
MLLW	0.00	-5.97

*PROJECT DATUM

0 50 100
SCALE, FEET

Attention:
0 1"
If this scale bar does not measure 1" then drawing is not original scale.

DRAFT

Designed:	---
Drawn:	DAP
Checked:	DJB
Approved:	DJB
P.E. No:	ME 13033
GEI Project	2004148

GEI Consultants
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Town of Bar Harbor
93 Cottage Street
Bar Harbor, ME 04609

Ferry Terminal Inspection & Assessment

121 Eden St.
Bar Harbor, ME

NO	DATE	ISSUE/REVISION	APP
2	2/8/2021	FINAL REPORT	DJB
1	1/11/2021	PRELIMINARY REVIEW	DJB

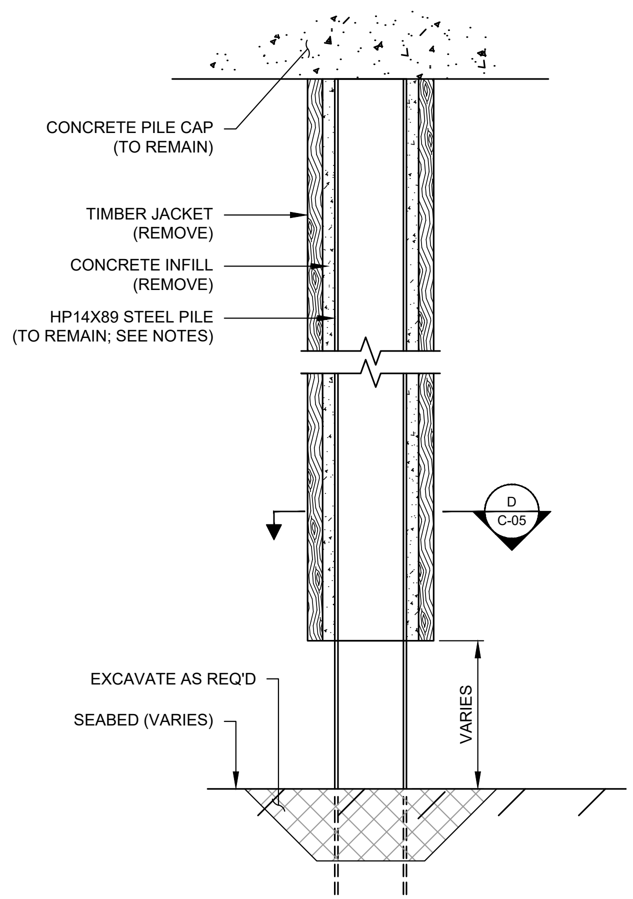
SHEET NAME
HYDROGRAPHIC SURVEY PLAN

SHEET NO.
V-01

1/8/2021 9:11:26 AM B:\Working\BAR HARBOR ME, TOWN OF\2004148 Ferry Terminal Inspection\00_CAD\Design\Working\Marina Concept Plans.dwg

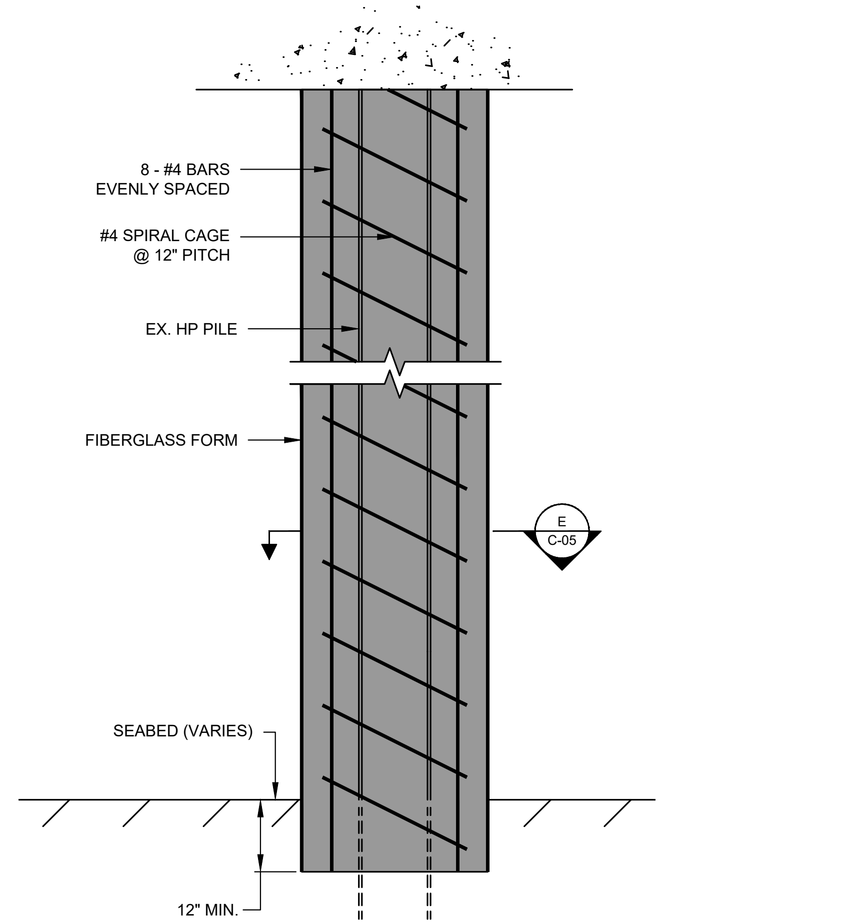
Appendix F

Repair Concept Plans

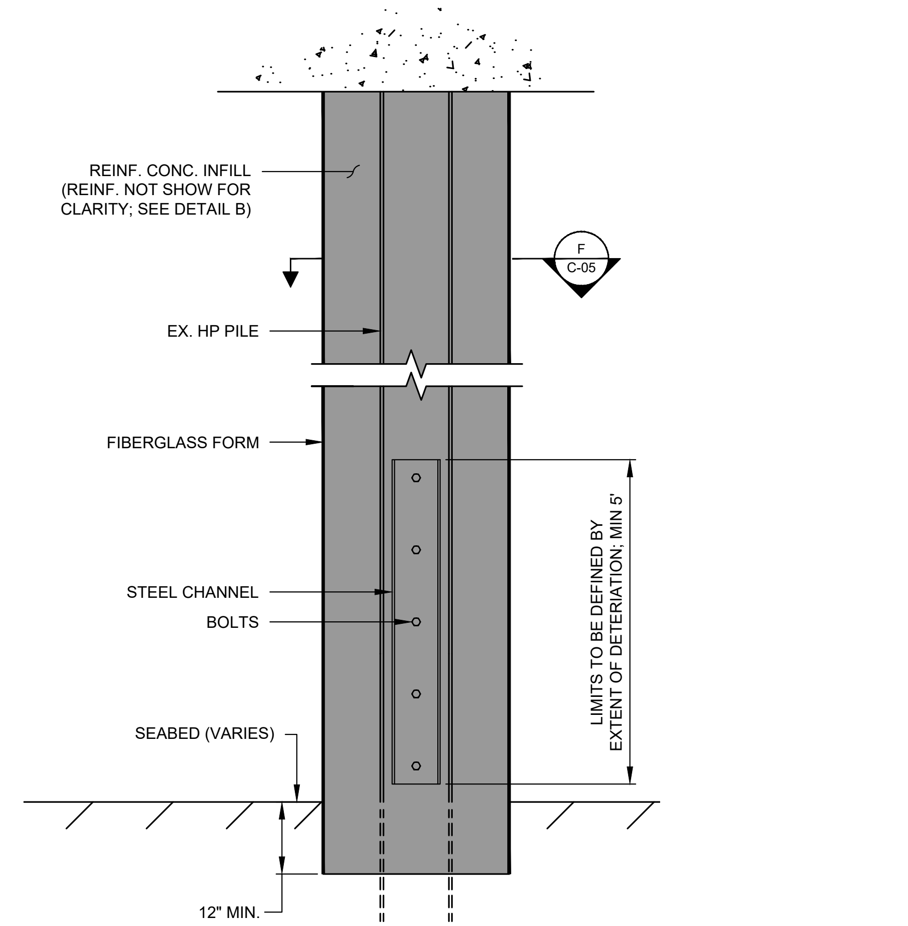


NOTES:
1. PILE SHALL BE THOROUGHLY CLEANED OF ANY EXISTING CONCRETE OR MARINE GROWTH.
2. FOLLOWING REMOVAL OF EXISTING JACKET AND CLEANING OF PILE, PILE TO BE INSPECTED BY ENGINEER TO IDENTIFY TYPE OF JACKET REPAIR REQUIRED.

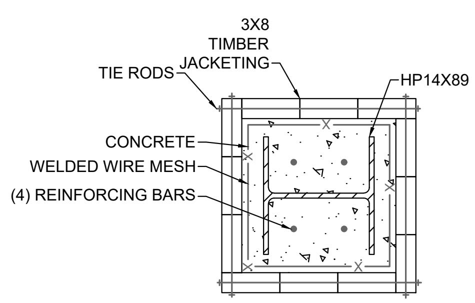
A EXISTING PILE ELEVATION
SCALE: 3/4" = 1'-0"



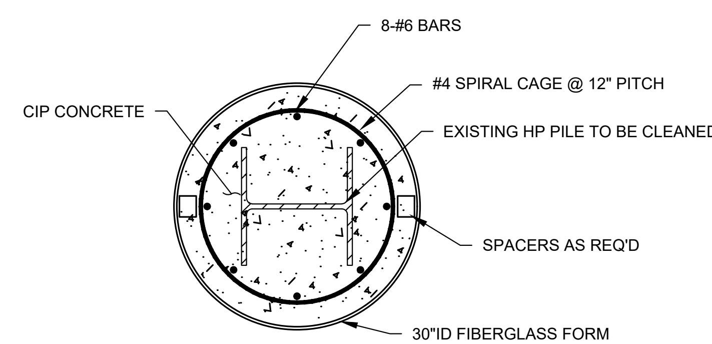
B NON-STRUCTURAL JACKET ELEVATION
SCALE: 3/4" = 1'-0"



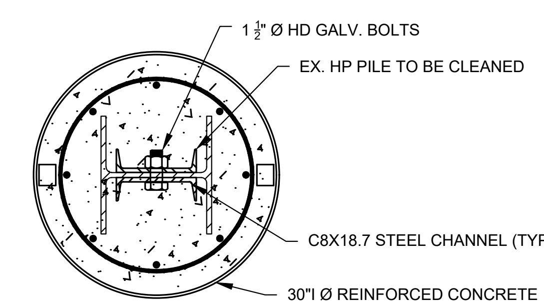
C STRUCTURAL JACKET ELEVATION
SCALE: 3/4" = 1'-0"



D EXISTING PILE CROSS SECTION
SCALE: 1"=1'



E NON-STRUCTURAL JACKET SECTION
SCALE: 1"=1'



F STRUCTURAL JACKET SECTION
SCALE: 1"=1'

PRELIMINARY

Attention: 0 1" If this scale bar does not measure 1" then drawing is not original scale.

DRAFT

Designed:	---
Drawn:	DAP
Checked:	DJB
Approved:	---
P.E. No:	---
GEI Project	2004148

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Bar Harbor, ME

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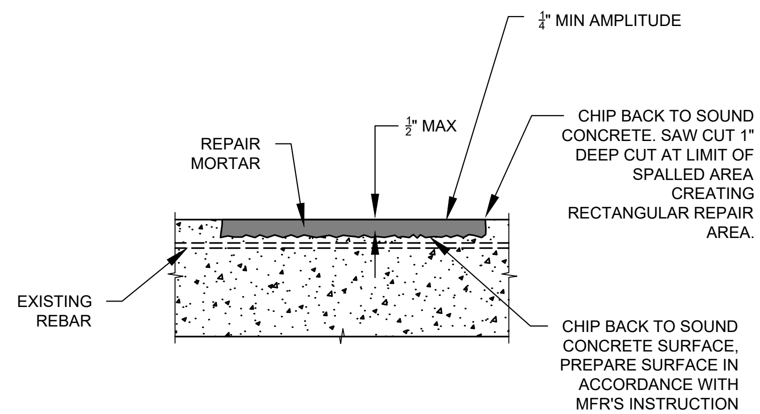
1	2/8/2021	FINAL REPORT	DJB

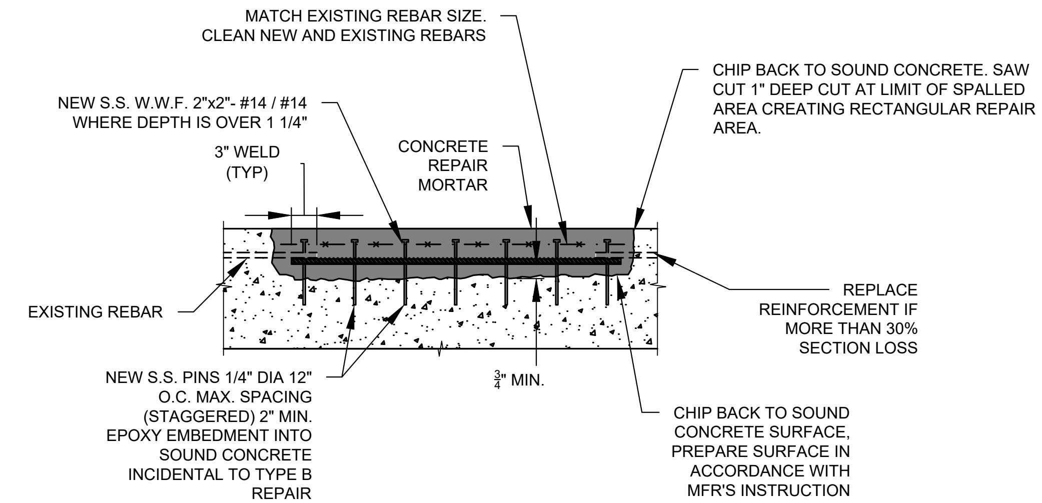
SHEET NAME
CONCEPTUAL PILE REPAIRS

SHEET NO.
C-04

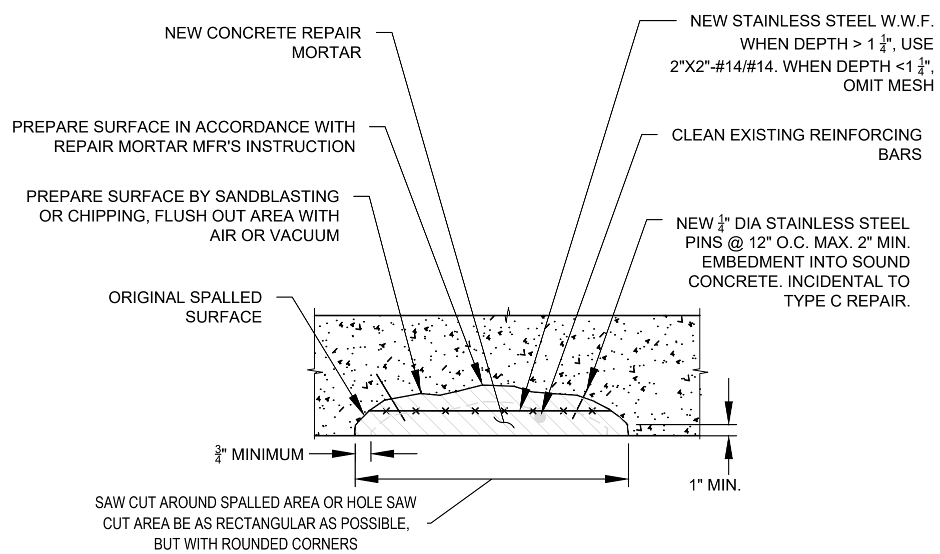
12/9/2020 11:55:16 AM B:\Working\BAR HARBOR ME, TOWN OF\2004148 Ferry Terminal Inspection\00_CAD\Design\Working\Repair_Options.dwg



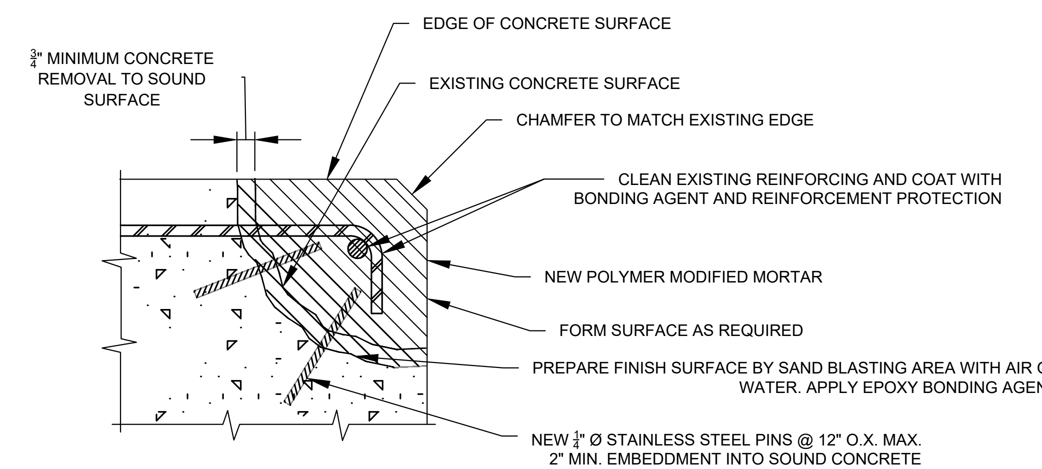
A DECK REPAIR TYPE A
SCALE: 1" = 1'-0"



B DECK REPAIR TYPE B
SCALE: 1" = 1'-0"



C UNDERDECK SPALL REPAIR TYPE C
SCALE: 1" = 1'-0"



D EXISTING PILE CROSS SECTION
SCALE: N.T.S.

PRELIMINARY

Attention:
0 1"
If this scale bar does not measure 1" then drawing is not original scale.

DRAFT

Designed:	---
Drawn:	DAP
Checked:	DJB
Approved:	---
P.E. No:	---
GEI Project	2004148

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Bar Harbor, ME

NO	DATE	ISSUE/REVISION	APP
1	2/8/2021	FINAL REPORT	DJB

SHEET NAME
CONCEPTUAL DECK REPAIRS

SHEET NO.
C-05

Appendix G

Marina Concept Design Memo and Plans

Memo

To: Mr. Charlie Phippen – Harbormaster

From: Daniel Bannon, P.E., CFM – Project Manager

c: Alan Pepin, P.E. – GEI Consultants
Bethany Leavitt – Public Works Director
Greg Johnston, P.E. – G.F. Johnston Consulting Engineers

Date: Revised Feb 8, 2021

Re: Marina Concept Design Alternatives
Ferry Terminal Inspection and Assessment Project
Bar Harbor, Maine
GEI Project No. 2004148

As an extension of GEI's current work on the Inspection and Assessment of the Ferry Terminal, the Town has requested that GEI prepare concept designs for redevelopment of the site as a marina for recreational or combined recreational and working waterfront use; considering options for repair and reuse or demolition and replacement of portions of the existing piers, and installation of new floating docks, gangways, boat ramp, and wave attenuation.

Three alternative concept plans have been developed for a new marina at the site. This memo describes the basis of design for the concepts. Plans provided in Appendix A include a Hydrographic Survey Plan, and three Concept Designs. Plans have been developed following the recommended criteria of American Society of Civil Engineers (ASCE) Manual of Practice 50 – Planning and Design Guidelines for Small Craft Harbors, and States Organization for Boating Access (SOBA) Design Handbook for Recreational Boating and Fishing Facilities.

The concepts, and disposition of the existing North and South piers, are summarized below:

Concept	A	B	C
Plan Sheet	M-01	M-02	M-03
North Pier	Short-Term Condition Repair and Reuse Long-Term Condition Demolish and Replace with Dolphins		
South Pier	Full Demolition	Repair and Reuse Inshore of Bent No. 7 Demolish Outshore of Bent No. 7	Repair and Reuse Inshore of Bent No. 10 Demolish Outshore of Bent No. 10

It is important to recognize that the work to date is conceptual in nature and intended to demonstrate a range of possible options and uses. None of the plans presented represent a formal proposal at this time. There are many possible variations on any of these concepts and the layout that is ultimately selected will be subject to further revision and refinement after uses have better defined and detailed analysis of the site exposure, mooring system, capacity requirements, operational needs, and other factors have been addressed.

Existing Site Conditions

GEI’s basis for understanding of existing site conditions includes recent above and underwater inspections of the facility, hydrographic and drone surveys completed by GEI (refer to Appendix A, Sheet V-01), and review of recent surveys, reports, and plans from a variety of sources as described in GEI’s Draft Inspection Report dated 11/13/2020.

Site Exposure

The site is on the eastern shore of Mount Desert Island with frontage on Frenchman Bay. The primary exposure is from the NNW to ENE directions, with a maximum fetch of approximately 5 miles from the north to northeast. The site is protected by Mount Desert Island, Bar Island, and The Bar from the northwest, west, south and southeast. It is anticipated that providing protection from the NNW to ENE directions will be critical to maintaining a functional marina. Protection out of the SE may also be beneficial. Future work should include a wind-wave analysis to better understand site-specific exposure conditions and the optimal layout and effectiveness of the wave attenuation system.

Vessel Types and Sizes

Given the uncertainty in future uses, the marina concepts have been developed for flexibility to accommodate a range of vessel types and sizes. Vessels considered in conceptual layouts include:

- North berth and existing RO-RO Ramp maintained for use by The CAT
- Recreational transient vessels in a range of sizes. Slips from 30’ to 70’.
- Based on federal Boating Infrastructure Grant eligibility requirements, minimum slips to accommodate 26’ vessels with 6’ of water at all tides.
- Several berths provided for occasional megayachts up to 140’ and infrequently up to 180’.
- Small cruise ships – e.g. American Constitution: 267’, or Independence: 225’
- Potential cruise tenders from larger cruise ships.

Draft Requirements

Minimum draft requirements for the marina are anticipated to range from 4’-6’ for typical small recreational vessels to 12-15’ for the largest megayachts that visit Bar Harbor. Water depth through most of the site is sufficient to support the anticipated range of vessel sizes. Location and layout of the most inshore floats needs to consider providing adequate water depth. The submerged rubble mound beneath/adjacent the existing north pier presents a potential hazard to navigation and provisions should be made to keep vessels away from this area.

Floating Docks

Main floats are presumed to consist of heavy concrete attenuator floats. 4m (13.12’) wide sections have been used along the North and (if included) South float runs. 6m (19.68’) wide sections have been used along the east run given the site exposure and potential for large vessel berthing. 3m (9.84’) wide sections are used where attenuator floats are not required. It is anticipated that all concrete floats will remain installed year-round.

Finger floats are anticipated to be seasonal and can be of lighter construction, potentially light-duty concrete, timber, or aluminum. Typical finger floats are 6’ wide, with lengths based on slip sizes.

Figure 1 – Concrete Wave Attenuator Floats installed 2020 in Provincetown, MA

Access

Access to the floats and primary berths must be ADA compliant. For a recreational marina with more than 25 slips, the minimum gangway length is 80’. For facilities serving cruise ships, the minimum gangway length is 120’. Due to potential use by small cruise ships, the more stringent criterion is anticipated to apply and is used in all options. This length of ramp is not only ADA compliant, but will provide a gradual slope that is friendly to all users, and fits well with existing conditions at the site given the distances required to extend from shore to deep water.

Boat Ramp

All concepts include a single-lane, 16’ wide boat ramp with a running slope of 15% that extends to approximately -4’ MLLW. This will allow for use at all tides by the majority of recreational vessels that would likely use the facility, but may present minor tidal limitations for some of the largest vessels. Side boarding floats are included for ease of access and loading. Options for constructing the ramp north or south of the south causeway are considered.

Mooring System

The new marina will require a mooring system to restrain the floating docks in position. It is anticipated that the mooring system will consist of large diameter piles, bottom anchors, or a combination of the two. The required mooring system has not been investigated during this concept planning phase and will be the subject of future detailed design efforts including a geotechnical investigation program for whichever approach is selected.

Reuse of Existing North Pier

The north pier is currently in a deteriorated state but is necessary to provide fendering to the north berth used by The CAT. In the short-term, it is presumed that the North Pier will remain in place with repairs completed as necessary to maintain safety and function. The length of time which the pier must remain in use will influence the extent of repairs required.

In the long-term, it is anticipated that the North Pier will be demolished and replaced with a system of dolphins. The dolphin system is anticipated to be far more cost-effective than a full in-kind replacement, and there is not believed to be a functional need for the large platform that would be provided by an in-kind replacement considering the current and future uses.

The short- and long-term conditions described above for the North Pier apply to all marina concepts considered.

Reuse of Existing South Pier

The three concepts that have been developed consider varying amounts of reuse or demolition of the existing south pier. All alternatives were laid out to not extend beyond the length of the north pier.

- Concept A would require that the south pier be removed entirely, which would provide a clean slate for redevelopment of the watersheet area.

- Concept B includes the demolition of the South Pier outshore of Bent 7, and the repair and reuse of the existing pier inshore of Bent 7. The result would be a rectangular pier approximately 32.5’ wide x 120’ long. Repairs would be required to the pier to address deteriorated conditions, however generally speaking this section of pier is in better condition than most other areas of the pier, and repair requirements would be less extensive. Additional upgrades to the pier would be needed including new fendering along the north and south faces, addition of edge protection, hoists, and other functional items that may support the future uses, for example, a marine pumpout, fuel service, or small commercial fishing pier unloading.

- Concept C includes demolition of the South Pier outshore of Bent 10, and the repair and reuse of the existing pier inshore of Bent 10. The result would be a larger pier with a pier head and turning area that could support greater volume of truck traffic. The pier head also allows for gangway access to the float system. Repairs would be required to the pier to address deteriorated conditions, and upgrades to the pier would be needed including new fendering along the south and east faces, addition of edge protection, hoists, coordination of gangway access, and other functional items that may support the future uses, for example, a marine pumpout, fuel service, or small commercial fishing pier unloading.

Reuse of the entire south pier was not considered because the layout of the existing pier is not compatible with the layout of wave attenuation necessary to protect from the primary exposed directions, is an impediment to navigation in the marina basin area, and functionally is not compatible with the proposed use as a recreational small craft marina.

Cost Estimates

Cost estimates have been developed for the three concepts for budgetary planning purposes. The costs are broken down into four overall categories:

- Demolition of existing pier components, which are broken down into initial demolition (Phase I) of the south pier and bridges and deferred demolition (Phase II) of the North Pier.

- Repair costs including pile and deck repairs of any portions of the structure to remain.

- Marina components, which includes floating docks, mooring and fender piles, gangways, fixed platforms, and ramps, boat launch ramp, and concrete abutments. Where appropriate, estimates are provided for the BASE and optional ADD-ON layouts.

- Marine dolphins for the long-term north berth, including associated catwalks and ladders.

Estimates are summarized in Table 1 below.

Table 1 – Marina Design Cost Estimates

Item	Concept #1		Concept #2		Concept #2
	M-01		M-02		M-03
	BASE	ADD-ON	BASE	ADD-ON	BASE
Phase I					
Demolition	$1,560,000	-	$1,350,000	-	$1,160,000
Repair & Upgrades	$0	-	$1,690,000	-	$2,830,000
Marina	$4,080,000	$1,010,000	$4,070,000	$560,000	$4,760,000
Phase II					
Demolition	$840,000	-	$840,000	-	$840,000
Dolphins	$2,080,000	-	$2,080,000	-	$2,080,000
Subtotal	$8,560,000	$1,010,000	$10,030,000	$560,000	$11,670,000
20% contingency	$1,710,000	$200,000	$2,010,000	$110,000	$2,330,000
Total	$10,270,000	$1,210,000	$12,040,000	$670,000	$14,000,000
Total BASE + ADD-ON	**$11,480,000**		**$12,710,000**		**$14,000,000**

Summary of Options

The three concepts are summarized in the following table based on a range of objective criteria. Reference should be made to plan sheet M-01 through M-03 that are appended to this memo for each of the concepts.

Table 2 – Summary of Options

Plan Sheet Reference	Concept #1 M-01	Concept #2 M-02	Concept #3 M-03
Wave Attenuation	Heavy Concrete Attenuator Floats Oriented to provide protection from NW to NE.		
Floating Docks	<u>Concrete Main Floats</u> (9) 13'x66' (1) 26'x33' (4) 20'x66' (1) 13'x39' <u>Finger Floats</u> (4) 6'x30' (9) 6'x50' <u>Ramp Boarding Floats</u> (7) 6'x30' (1) 6'x26' *<u>Optional Addition</u>* *(5) 13'x66' concrete* *(3) 6'x70' fingers*	<u>Concrete Main Floats</u> (8) 13'x66' (1) 26'x33' (4) 20'x66' (1) 13'x39' (2) 13'x33' <u>Finger Floats</u> (4) 6'x30' (9) 6'x50' <u>Ramp Boarding Floats</u> (7) 6'x30' (1) 6'x26' *<u>Optional Addition</u>* *(3) 13'x66' concrete*	<u>Concrete Main Floats</u> (9) 13'x66' (1) 26'x33' (3) 20'x66' (1) 13'x39' (4) 10'x66' (2) 13'x51' (1) 13'x37' (1) 20'x49' <u>Finger Floats</u> (3) 6'x70' (7) 6'x35' <u>Ramp Boarding Floats</u> (7) 6'x30'
Total Float Area	24,700 SF	21,550 SF	22,350 SF
Slips / Berths	(9) 30-ft (18) 50-ft (1) 290-ft *Optional Addition* *(8) 70-ft* *(1)78-ft* *(1) 263-ft*	(9) 30-ft (18) 50-ft (1) 214-ft (1) 290-ft *Optional Addition* *(1) 178-ft* *(1) 180-ft*	(14) 35-ft (1) 65-ft (7) 70-ft (1) 260-ft (1) 460-ft
Gangway Access	120' ADA compliant gangway Located on upland in between north and south causeways	120' ADA compliant gangway Located on upland in between north and south causeways	120' ADA compliant gangway Located at head of fixed pier
Mooring System	To be determined		
Boat Ramp	Located on north side of south causeway	Located on north side of south causeway *Alternative option to locate on north side*	Located on south side of south causeway
Fixed Pier	None	3,900 SF (2) 90-ft berths Hoists and ladder access to vessels	7,360 SF (1) 70-ft berth (1) 140-ft berth Hoists and ladder access to vessels Gangway access to floats
North Pier	<u>Short-term Condition</u>: Retain North Pier with minor repairs <u>Long-Term Condition</u>: Demolish and Replace with Dolphins		
Total Cost (USD in Millions)	$10.27 Base + $1.21 Add-on = $11.48 total	$12.04 Base + $0.67 Add-on = $12.71 total	$14.0 total

DJB:ADP

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PRELIMINARY

ELEVATION	MLLW*	NAVD88
BFE	+22.97	+17.00
HAT	+13.31	+7.34
MHHW	+11.37	+5.40
MHW	+10.94	+4.97
MSL	+5.67	-0.30
MLW	+0.38	-5.59
MLLW	0.00	-5.97

*PROJECT DATUM

0 30 60
SCALE, FEET

Attention:
0 1"
If this scale bar does not measure 1" then drawing is not original scale.

DRAFT

Designed:	DJB
Drawn:	DAP
Checked:	DJB
Approved:	DJB
P.E. No:	ME 13033
GEI Project	2004148

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121 Eden St.
Bar Harbor, ME

NO	DATE	ISSUE/REVISION	APP
2	2/8/2021	FINAL REPORT	DJB
1	1/11/2021	PRELIMINARY REVIEW	DJB

SHEET NAME
MARINA CONCEPT PLAN A

SHEET NO.
M-01

PRELIMINARY

ELEVATION	MLLW*	NAVD88
BFE	+22.97	+17.00
HAT	+13.31	+7.34
MHHW	+11.37	+5.40
MHW	+10.94	+4.97
MSL	+5.67	-0.30
MLW	+0.38	-5.59
MLLW	0.00	-5.97

*PROJECT DATUM

0 30 60
SCALE, FEET

Attention:
0 1"
If this scale bar does not measure 1" then drawing is not original scale.

DRAFT

Designed:	DJB
Drawn:	DAP
Checked:	DJB
Approved:	DJB
P.E. No:	ME 13033
GEI Project	2004148

GEI Consultants
GEI CONSULTANTS, INC.
5 MILK STREET
PORTLAND, ME 04101
(207)797-8901

Town of Bar Harbor
93 Cottage Street
Bar Harbor, ME 04609

Ferry Terminal Inspection & Assessment

121 Eden St.
Bar Harbor, ME

NO	DATE	ISSUE/REVISION	APP
2	2/8/2021	FINAL REPORT	DJB
1	1/11/2021	PRELIMINARY REVIEW	DJB

SHEET NAME: MARINA CONCEPT PLAN B

SHEET NO.: M-02

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PRELIMINARY

ELEVATION	MLLW*	NAVD88
BFE	+22.97	+17.00
HAT	+13.31	+7.34
MHHW	+11.37	+5.40
MHW	+10.94	+4.97
MSL	+5.67	-0.30
MLW	+0.38	-5.59
MLLW	0.00	-5.97

*PROJECT DATUM

0 30 60
SCALE, FEET

Attention:
0 1"
If this scale bar does not measure 1" then drawing is not original scale.

DRAFT

Designed: DJB
Drawn: DAP
Checked: DJB
Approved: DJB
P.E. No: ME 13033
GEI Project 2004148

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NO	DATE	ISSUE/REVISION	APP
2	2/8/2021	FINAL REPORT	DJB
1	1/11/2021	PRELIMINARY REVIEW	DJB

SHEET NAME
MARINA CONCEPT PLAN C

SHEET NO.
M-03

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