City of Rockland
Downtown Waterfront Marine Infrastructure
Facility Assessment

Rockland, Maine
October 31, 2022

Prepared for:
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1. INTRODUCTION

Project Background

The project builds on the momentum of a successful planning, conceptual design and community engagement effort for the Downtown Waterfront that has developed a compelling vision for the City's infrastructure as a connected, functional, and resilient asset meeting diverse community and regional needs. This work was guided by the ongoing Ad Hoc Downtown Waterfront Advisory Committee that has gained consensus on resiliency goals and a balance of varied uses and stakeholders. A copy of the Downtown Waterfront Concept Plan is included as Attachment A. The logical next step is preliminary engineering of the waterfront infrastructure. This assessment will begin that process.

A detailed visual inspection of the existing marine infrastructure was conducted to document and evaluate the existing conditions, including the structures' susceptibility to damage or failure given the location, design, age, condition, and/or state of repair. Resiliency improvements and upgrades are identified to be considered during preliminary design. This work will leverage the Maine Coastal Program-funded work already undertaken by Wood Environment & Infrastructure Solutions, Inc., as well as that undertaken by the project team during the conceptual design process. As part of this evaluation, the design team evaluated opportunities for sustainable building materials and green infrastructure within the project area.
The areas of assessment included:

- Public Landing
- Harbor Park Seawall
- Middle Pier/Buoy Park

Recent survey data was collected and these assessment areas are depicted on an Existing Conditions Plan. Note that the elevations depicted are based on Mean Lower Low Water (MLLW) datum and flood elevations are generally referenced to the NAVD88 datum, which 5.68' higher, in Rockland. The MLLW datum is used in marine charts and is more useful for marine projects since it depicts working conditions at a low water condition. This plan is included in Attachment B.

2. FACILITY ASSESSMENT

In 2016, A Summary Report on The Rockland Public Landing was prepared by Milone & MacBroom, along with Landmark Corporation and Pinnacle Hill Engineering. This Report has been quoted as a starting point for our current assessment and recommendations. In each area, this will be noted as "the 2016 study" and will be set off by quotation marks. It is deemed unnecessary to include the full report. In addition, Wood Environment & Infrastructure Solutions, Inc. performed a Vulnerability Assessment and Resilience Planning for the Middle Pier. We have also quoted this study as "the 2019 study" and again set off the findings in quotation marks. We have included the full report as Attachment C. Our current findings are included in bold text to distinguish them from the 2016 and 2019 studies.

2.1 Public Landing

Float Assessment

An assessment of the public landing floats was made in the 2016 study when the floats were out of the water in winter storage. The following is an excerpt from that study:

"The existing floats appear to be all the same type and design with good freeboard, details, and generous dimensions. If they were to be replaced, the new floats would probably be the same type and construction. The floats are of a modern design that is an industry standard in the United States. These floats appear to be the type made popular by Custom Float Services in Portland Maine, which utilizes pressure treated southern pine lumber, ACE-brand polyethylene foam-filled float drums, and galvanized connection hardware and bolts. Custom Float Services sells these floats made to order or sells plans and components if the customer wishes to build them. Given the size of these floats, they were probably constructed locally.

It appears that the City is maintaining the floats as needed. In previous work, our engineers observed ongoing work on several of the floats to replace the skids. With the large number of floats and the age of some of them, it is expected that there will be some floats each year that will need parts replaced.
While the inspection team was favorably impressed with their conditions and maintenance, they noted a need to examine the floats again once in the water.

The float construction qualities were noted as follows:

- The decking is 5/4x6 southern pine decking boards.
- The main deck framing spans the short direction of the floats and bears directly on the float drums. The examined floats have 4x8 southern pine at about 24" on center spacing. Joints and hinges are galvanized steel fabrications with galvanized bolts.
- The typical float drums are the ACE Polyethylene 48 x 72 x 20" deep. Most of the floats are 10ft x 30ft with six float drums under them

The floats are of two, or possibly three, age groups. Some are virtually new. The older group of floats are still in relatively good condition, but have some deterioration, primarily in the skids, horizontal beams, and skid vertical struts. These are the portions of the wood understructure extending below the float drums and thus always in the water and always submerged. The skid structure's only function is that it protects the float drums when they are dragged across the ground. They could not be dragged if there were no skids since the float drums have very thin plastic thickness, about 1/8" to 1/4" usually. Nationally, most marina floats do not have skids, and thus do not have skid maintenance. In areas where the floats can be left in the water year-round, this makes good sense. In the North, where floats are removed in the winter, care must be taken to lift the floats in and out of the water, or to have the skids so that they can be dragged; it becomes a trade-off.

Presently, the older float skids have deteriorated and weakened, and thus are easily damaged when dragging the floats out of the water. The examination of the floats revealed that there has been decay of the wood, and corrosion of the steel fittings and fasteners in the skid portion of the floats. In this weakened condition, it is understandable that dragging them would cause further damage. The Harbor Master at the time indicated that he intends to use a crane with spreader bars for lifting them out of the water to minimize damage in the future.

Without taking a detailed inventory, it appears that something around 50 % of the floats have deteriorating skids that will need replacement in the near future. In these cases, replacement of the entire float does not appear to be justified since the majority of these float structure is sound. Replacement of the entire skid would be easier, and more successful rather than repairing it piecemeal.

The 5/4 decking seems generally in good condition and is attached with stainless steel screws. Weathering has occurred on some of the older floats. Some floats have had partial replacements of the deck boards, and the inspection team did not see anything that is generally unsafe. Continued maintenance of decking will be necessary. Sometimes it is possible to simply turn the boards over and refasten them, giving a virtually new and un-weathered surface, if the boards are otherwise still structurally intact.

CCA pressure treated lumber is required for wood that is constantly submerged in salt water. The pressure treating industry has veered away from CCA, with a voluntary program to eliminate selling it in the homeowner and light commercial markets. Nowadays, the pressure treated lumber available for purchase in normal lumber yards will be one of the non-CCA types, although it will be the same in appearance and color. These alternatives do not stand up in salt-water submergence. Thus, a new marine structure generally contains a mixture of preservative types: CCA for submerged timber and piling, and non-CCA for above waterline members, such as decking and handrails where human contact is expected. In the Rockland floats, the decking exposed to the public may be that mixture, but given the age of some floats, the older ones may have the CCA type. It is expected that any of the newer floats would have the alternate types of treatment.

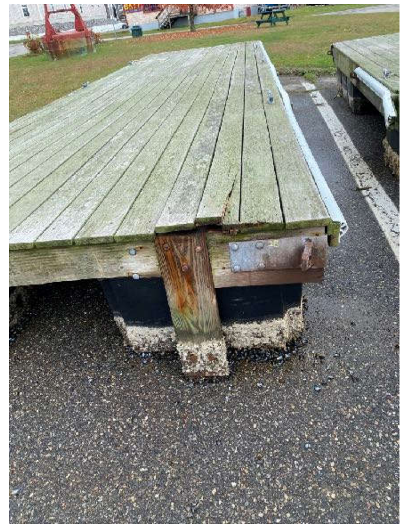
Any boards with splintering should be replaced as soon as noticed to avoid splinter hazards for people walking with bare feet.

When replacing the submerged skid timbers, it is important to purchase CCA treated lumber. This is the only treatment that is effective in salt-water submerged members. If purchasing small quantitates at local lumber yards, the City will need to be careful to specify and obtain CCA since that is no longer normally stocked in lumber yards for terrestrial construction. It is still available to marine contractors and others on special order, or through marine timber suppliers.”

We recently conducted an assessment of the public landing floats (10-26-22) when a portion of them were removed from the water:

<u>Floats</u>
We met and discussed the floats with Molly Eddy, Assistant Harbormaster. Around 20 floats were still in the water and about 12 were on shore. The City has a good numbering system and

mapping of the float positions that allows the floats to be returned in the spring to the same spot and to fit into the guide pile system. At this time, it appears that a more proactive program for replacement or rehabilitation would be desirable, as the City is not keeping up with the deterioration of the floats. In particular this year, a large number of the floats out of the water are missing the skids (see picture), which have failed from decay of submerged wood and corrosion of the associated bolts.

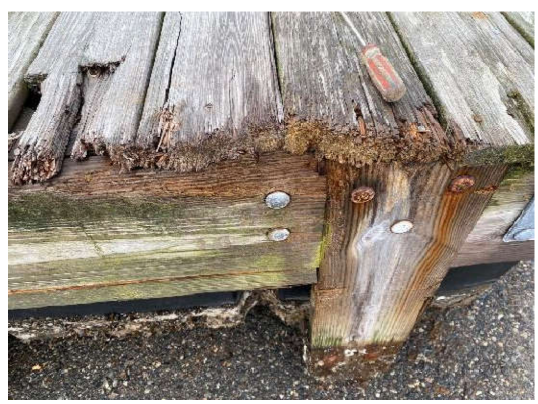


Other observations:

- **All floats are unpainted**
- **Various types and configurations of collars and anti-friction devices are present, but some have no provision to alleviate chafe**
- **The floats are of multiple age groups**
- **The older floats are noticeably more deteriorated, primarily in the skids- both the horizontal skid beams, and skid vertical struts**

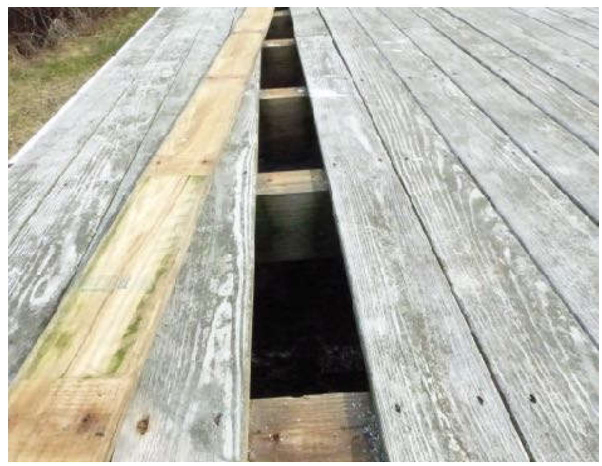
If floats are removed in the winter, care must be taken to lift the floats in and out of the water, or to have the skids so that they can be dragged. Lifting the 30 ft-long floats would require a crane or loader with a special spreader bar, since they probably cannot be lifted by a sling around the middle of the float. Once there has been decay in the wood or corrosion of the steel fittings and fasteners, the float skids are easily damaged when dragging the floats out of the water. In 2016, without taking a detailed inventory, we reported that it appeared that around 50 % of the floats had deteriorating skids, but they were being addressed. This year, the skids are actually missing from or heavily damaged from a large fraction of the 12 floats that had been hauled.

We also noted damage on a number of floats in the areas where they contact the guide piles, some severely (see picture).



The ends of the deck boards at the gap between floats is ragged in many cases and should be repaired (see picture). The gap dimension between the floats varies in width. The City has plywood transition strips covering the gap on some, but not all, floats. Those seem to work, but could be tripping hazards since they are uneven and worn. Molly Eddy thought that the floats worked better where they had added rigid lumber strips to each float to narrow the gap. This appears to be a better and safer solution.

Replacement of the entire floats does not appear to be necessary, since most of the float structure is sound. Rehabilitation would involve removing the floats from the water, pressure washing as necessary, removing the deck boards, inspecting the timber frame, and replacing anything that is suspect. Replacing fasteners, bolts, and reinstalling the decking. If the boards are otherwise still structurally intact, it is possible to simply turn the boards over and refasten them, giving a virtually new and un-weathered surface. Fasteners should be new stainless steel deck screws or equivalent.



We would recommend that you consider flipping the boards over as a part of a general rehabilitation. The top surface of the deck boards is weathered on all of the floats, but it can be seen that the bottom surface of the boards is like new (see picture). The weathering is not usually significant for weakening the boards, but harbor dirt and bird droppings make cleaning difficult, as well as the poor appearance. The 5/4" decking seemed generally in good condition in 2016, but in 2022 seems to be more weathered and deteriorated, especially on some of the older floats. Some floats have had partial replacements of the deck boards and our inspection did not show anything that is generally unsafe, though continuous inspection and maintenance of decking will be necessary. Some of the floats have nailed decking with fairly small common nails which are corroded and have lost the nail heads, thus releasing the boards. It is noted that many marina floats are painted to preserve the lumber surface. That would be a big change for Rockland if that were considered, but would be an upgrade in the appearance, at the expense of more maintenance.

Two floats on shore that have composite decking which seems to have performed poorly. The tie rails and rub strips have especially deformed and broken and should be replaced with stronger wood tie rails and plastic fendering.

Summary of Floats Condition

The floats appear to be in generally fair condition and will be usable for many more years, with increased maintenance. There will be continuous annual repair work required as the floats age. We feel that the work required is getting ahead of the City and recommend increasing that effort on an annual basis. Many of the floats are of similar age, which causes their repair needs to come at once.

Any plans for reconfiguring the marina can and should make use of these floats. Assuming they will be re-used and not replaced, we recommend that the City increase the pace of maintenance and rehabilitation of the floats. This is a matter of some study and cost analysis as the best way to accomplish that. The City may wish to purchase new floats or rehabilitate the older ones and in either case can do this by outside contractors or by work with in-house staff.

We recommend that a clear policy be made to either drag them on skids or lift them out and that the lifting equipment be acquired, or skid maintenance be done to support that decision.

Float Pilings

When we inspected the floats in 2016, they were out of the water and we could not see the floats in position on the guide piles. This year, many of the floats of the system were still in the water. This allowed us to examine the float piles and how the floats slide on the guides or lack of guides.

In general, the large outer float on the southeast side appears to have a good system of three pile dolphins and single piles that appear recent. The dolphins have one greenheart plumb pile and two southern pine bracing piles. The greenheart piles are much more chafe resistant than southern pine and they appear to be performing the best of the plumb piles. All floats need protective plastic anti-friction slide blocks on the float area that is in contact with the pile.

Other guide piles in the system are wearing badly, particularly the southern pine single piles or several that have floats with metal edges or sharp-edged wood members. In those cases, the wear is up to 50% of the original pile, or the float is damaged, or both. The main outer floats on the southwest side have very poor aluminum bent tube collar system. The base plates are shaving the piles, with a substantial pile of sawdust clearly visible during our visit (see picture). Those should be reconfigured and fitted with plastic slide blocks to protect the pile.

Redesign of the float system would offer the opportunity to remedy some of these problem areas; coupled with constant attention thereafter to wear points and reducing stresses in the connections.

Existing Pier Visual Examination

An assessment of the public landing piers was made as part of the 2016 study. The following is an excerpt from that study:

- "The Public Landing main pedestrian access is a steel, concrete, and granite bridge structure that starts on the seawall adjacent to the Harbormaster's office, and extends 142ft outward from the wall, with the width of the concrete walk surface deck of 12'-

3". The structure consists of two steel bridge spans with concrete deck, and two granite quarry stone dry laid pier structures in the harbor.

- The granite piers are solid, long-lasting structures and have no real issues that have been visible. The truss steel bridge is deteriorating and should be replaced or repaired. The outer steel span is also corroded and needs repair.
- Both steel spans should have structural evaluations and detailed analysis for load capacity. Some repairs may be needed to continue using them. Since the engineers were not familiar with what structural evaluations have been done, they recommended in 2016 that the City investigate records to determine whether there have been any recorded and take additional steps, if necessary.
- The small wood cabin on the outer pier appears to be functional, but is shabby and could stand to be renovated or replaced to improve aesthetics."

Existing Steel Truss Bridge Visual Examination

In 2016, Engineers observed the following in their on-site inspections of the steel truss bridge portion of the pier:

- "The steel truss is a distinctive bridge spanning about 65ft x 12'-3" deck width. It has been reported that the bridge was formerly part of the Rockland local railroad system used in quarrying limestone and was repurposed as a part of the pier at the public landing.
- Railroad bridge loading is generally much higher than pedestrian or light vehicular traffic. While there were a wide range of railroad design loads, it is likely that the bridge was over designed for the usage now.

- Trusses: The Bridge consists of two riveted steel trusses 8ft deep x 65ft long, bearing on heavy-cast steel or iron bearings, and supported on the granite walls at each end. The trusses are the main load-carrying elements and have been well painted and cared for. There seem to be few signs of corrosion except a few localized dents and some light corrosion on the bottom members near the ends. These trusses have escaped the corrosion seen on the other members underneath, perhaps because they are exposed to the rain, which would wash off any salt spray deposits.

- The trusses consist of 8 equal spaces where there is a vertical member and connections to the X bracing each vertical, there is a cross beam under the deck that spans between the two trusses. This beam is a riveted beam, made up of a vertical web plate and two angles on the top and two angles on the bottom. The overall assembly is 20 1/2" deep x 8 1/2" wide. These span about 13'-2". These beams are heavily corroded and have lost significant original capacity, though they were very heavy to start with. Most of the corrosion is on the bottom flanges and is likely caused by splash and spray of salt water. The solid deck overhead would prevent rainwater from washing it off. In some cases, the angles of the bottom flange of the beams have nearly disappeared.
- Running perpendicular to the riveted beams and spaced 6'-0" apart are two rolled steel beams. These beams are 15 1/2" deep x 5" wide and span only 8ft between the riveted beam. As a railroad bridge, these would probably have been more or less directly under the tracks, though it is hard to say since railroad track gauges varied a great deal. These beams now support only the concrete deck slab and seem oversized. Corrosion has damaged these beams as well, though not to the same extent as the cross beams. It appears to be a loss of up to 50% of the bottom flange in several places.
- Concrete deck slab: This appears to be a cast-in place slab that was constructed when the bridge was converted to a pier. The slab is thick and fortified with steel reinforcement. There has been corrosion of the reinforcement on the bottom side that has caused the surface to split and fall away. This exposes the reinforcement to additional corrosion.

Eventually, the concrete must be replaced. The worst area seems to be close to the shore seawall, no doubt due to waves splashing upward after striking the wall. Other parts of the slab show no corrosion spalling.

- X bracing is under the bottom of the beams and consisting of square steel bars with heavy clevis end connections to the trusses. The square bars are heavily corroded and bent. Some have reached a point where they will probably soon break. They catch the eye as the most visible of the corroded members. As a truss bridge for a railroad, the original bridge needed extensive bracing between the trusses to handle lateral loads from the trains and wind. That role can be handled by the concrete deck slab, however and the existing X bracing can be discarded rather than repaired."

Existing Steel Beam Bridge Examination

Engineers observed the following in 2016 as part of their on-site inspections of the steel beam bridge portion of the pier:

- "The second outer bridge span is a much simpler steel bridge consisting of two steel beams with deck panels that span between them. The beams are rolled steel I-beams 24" deep x 7" wide. These steel beams have corroded on the bottom flanges visibly, mostly on the inside, not visible except from below.

- They have lost some steel cross section of the bottom flanges. Perhaps up to 25% in several places.

- There is one very corroded and bent diagonal that should be replaced. In this span, the concrete slab appears to be a precast slab and would not provide lateral bracing of the span unless it was designed into the connections. It appears that this was part of an original bracing system, there may have been similar diagonals in the other bays that have already fallen away.

- The concrete deck slabs appear to be newer and are supported on widely spaced galvanized steel square tubes. There is light corrosion on the galvanized tubes. The concrete slab looks fine from below.

- The head platform of the new 80ft gangway installed in 2015 was bolted to the side of this span."

Visual Examination of Granite Piers

In 2016, Engineers observed the following in their inspection of granite piers:

- "There were no visible issues noted during the pier inspection. A large rusting block on the seaward outer side has an unknown purpose, and the City should consider its removal. The City should photograph the pier from all sides during an extreme low tide to document its current condition below normal water level.

Load Rating & Repairs

While the bridge was formerly a railroad bridge and probably has quite a large safety margin, our engineers recommend that the City undertake a load rating.

After a first look at the pier steel framing, engineers noted that the riveted beams crossing the bridge between the trusses are the main concern and repairs should be undertaken soon. These beams could be strengthened by burning off the rivets and removing the two bottom angles and welding on new angles. Engineers were not able to ascertain the extent of framing in this condition."

We recently performed an assessment of the Public Landing Pier, Steel Truss Bridge, Steel Beam Bridge, and Granite Piers (10-11-22):

- **There has been no visible repair work on the pier structure since the 2016 inspection.**
- **Visible corrosion has not advanced significantly, but previous concerns still exist.**
- **The scope of this preliminary design report does not include remedial repairs, since the proposed project will involve demolition of this structure.**
- **A new 80ft gangway has been installed since the prior assessment. We noted that this gangway has damaged mesh side panels near the lower end. The transition plate at the lower end has a worn bottom edge and needs replacement of the anti -friction wear strip.**
- **The older existing aluminum gangway on the south end of the public pier has significant damage in the form of bent railings and should be repaired or replaced. The railings form a part of the structure, a structural truss on each side, and it is compromised by the damage. Heavy loading with multiple people on this gangway during events should be restricted until it is repaired. [Note: Preliminary design does not include re-use.]**
- **The small gangway leading to the dinghy dock is very short for the tide levels in Rockland and at low tide is very steep. During this year's inspection of the gangway, we noted that it is hanging from the upper hinge with no float in place under it and was submerged in the water at the lower end. Hopefully it can be removed from the water quickly. [Note: Preliminary design does not include re-use.]**
- **The granite pier shows no sign of deterioration or displacement, but removal of rusting block and photo documentation is still recommended**
- **The small dock attendant's office has been replaced or repaired**

2.2 Harbor Park Seawall

Excerpt from Wood Environment & Infrastructure Solutions, Inc. Assessment from the 2019 report (see Attachment B for full report):

"Shoreline protection at this location was not directly assessed however appears to be a stacked granite sea wall which may also support the wharf. Delamination of material was not observed, and structure appears to function as intended."

We recently assessed the Harbor Park Seawall (10-11-22):

The seawall is the dry-laid quarried stone typical of coastal New England. Granite wharves of the same construction are generally long-lived. The main length of the seawall is in good condition and has been updated with a concrete cap as a sidewalk and vehicle barrier, and an aluminum handrail.

Based on a Mean Lower Low Water datum, the sidewalk level varies from about El. 14.5 at the east end near The Pearl restaurant, rising to El. 17.2 at the Public Landing and Harbormaster Office. In order to address Sea Level Rise, it has been determined that the wall height should be 18.4 ft, an increase of 1.2 to 3.9 feet.

The mudline at the toe of the wall is generally uniform at about +1 ft , so the wall face exposed is approximately 12 to 16 ft high. The proposed project will be increased to a uniform height of about 17 ft.

The City-owned portion of the seawall is about 365 feet long and forms a gentle curve along the Harbor. One short section of about 17 ft, between the City Public Landing and the Boston Financial boardwalk was never repaired with the concrete cap. That section should have the same treatment to match the Boston Financial configuration and form a continuous boardwalk with the new work proposed for the City Seawall. The wall under the green steel bridge, about 13 ft in length, should also be capped and modified for the new pier when this work is undertaken, and the bridge is removed.

The seawall, as visible, generally looks stable and the masonry wall face is fairly straight and planar. The top several courses of stone are especially wavy and exhibit a great deal of poor workmanship in the fit of stone and in the joints, but stones have been locked into place when the concrete sidewalk was placed.

Several storm drainage outfalls penetrate the seawall to discharge in the harbor. The Engineers and the City should coordinate on any required modifications to the existing outfalls when preparing the construction documents.

2.3 Middle Pier/Buoy Park

Excerpts from Wood Environment & Infrastructure Solutions, Inc. Assessment from the 2019 report (see Attachment B for full report)

"The wharf structure, particularly the stacked granite foundation, exhibited no apparent signs of translation or dislodgment for the large members. Smaller stone material was noted between the surface and granite blocks, observable through the large openings between the blocks. Above this area on the pavement, large cracks and dips are observed. Previous repair work was also noted at some locations. As earlier mentioned, an abutment is used to support the pier framing at one end. Positive attachment (anchorage or other mechanical fastener) of the concrete abutment to the granite blocks or attachment of the wood framing to the abutment and/or the granite blocks could not be confirmed. The abutment stem and footing appear to be composed of two separately poured elements. Wooden members, which include piles, stringers, and decking exhibit signs of moderate deterioration due to weathering and/or microbial attack. Some minor to moderate conditions of shakes, checks and splits were observed throughout. For those piles observed, signs of infestations such as marine borers were not noted in the tidal zone.

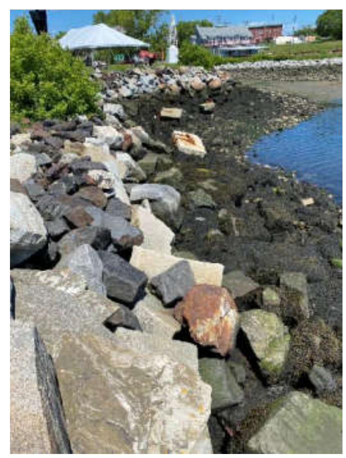
Based on the present-day model for BFE of 11 to 15 feet which includes a wave height of 2 to 6 feet, the pier substructure and decking would be potentially impacted by high velocity wave action. In the case that any elements such as the abutment, stringers, etc., are not positively attached to subsequent load carrying members, dislodgement or delamination of material at the top of deck should be expected. Otherwise we would not expect any impact beyond minor delamination at the deck. A granite block or other material appears to be missing which would provide transfer of bearing loads from the pier. We were unable to view and assess other locations of the pier and wharf to note similar conditions at other locations due to limited access around the pier, which was provided only via the floating dock. The wharf can be expected to experience loss of smaller diameter crushed rock at the sub-base from washout resulting in deflection of the pavement and possibly complete delamination. Similar behavior of the pier and wharf can be expected for future floor scenarios and the possibility of impact more likely as the return period for conditions representing the present day BFE decreases. Site utilities which include water and electricity are exposed to wave action and inundation at the pier and the floating dock. A timber pipe bridge, which supports an electrical conduit, is located in the plane of the deck and near the floating dock entrance. The structure does not appear to be securely fastened or designed to resist impact from wave action. In addition, the electrical cabinet and conduit at the wharf do not appear to be of waterproof construction. Given the current position of the cabinet, we would expect some exposure to waver during the BFE for the Mid Term Scenario.

The floating dock assembly consists of the gangway and the floats. The gangway attachment allows for rotation with a maximum limited by the elevation of the float at or beneath hinge elevation. Normal operation does not appear to be influenced by the MHHW for all scenario. However, for the Short Term scenario, the gangway will be subjected to wave loading and uplift forces from the Stillwater elevation. In addition, the floats are moored to the perimeter piles at the south side of the pier via mooring chains. This attachment allows for a maximum mooring elevation roughly 15 inches below the top of pier deck. Estimating some flexibility in this connection, the dock will be limited from traveling beyond elevation 9.5 feet (9 ft 6 inches) and begin to exert loading on the pier at water levels above this elevation.

Shoreline protection is provided by a revetment ranging in elevation from about 11 feet to 12 feet. Large diameter (roughly 1.5 to 4 ft) riprap is provided along the perimeter of the site extending from below the low tide level to the top of grade. The estimated slope is a maximum of 3 to 1, horizontal to vertical, and gradation appears to be suitable based on condition of slope. No signs of material degradation or slope instability or piping were noted. Based on existing conditions, the risk of overtopping during the Present Day scenario is relatively low. Overtopping is more likely for the Short and Mid Term scenarios. Some landward flooding in the range of 2 to 7 feet will occur during the overtopping during these scenarios but it will not undermine the revetment. Under wave attack, randomly placed riprap will experience some settlement and readjustment; however, the risk of wide-scale riprap slope failure appears low. The risk of localized scour or dislodging of riprap is low and given their inherent stability they will likely require minimal remediation for the Short Term and future scenario."

We recently performed an assessment the Middle Pier (10-11-22):

The following assessment is based on a visual review of the Middle Pier from above and from the adjacent float system. No destructive removal was done to access hidden components or materials and no borings or other testing was performed.



The shoreward portion of the Middle Pier and Buoy Park complex starts at the street and runs southward toward the harbor encompassing a lawn and paved areas, including several small structures, the food truck areas, and buried and above ground structures for the Rockland sewer system. Buoy Park offers a large lawn area with antique navigation buoys on display. The entire area appears to be a manmade filled embankment. Both east and west edges of the embankment are slopes with stone rip rap and vegetation. These slopes appear to be stable and since limited to no modification of these slopes is contemplated, the embankments were not examined in detail.

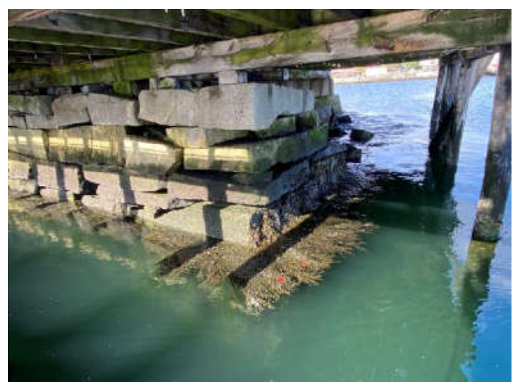
A survey was performed by Landmark in 2022 for this report and design, including bathymetry from 2018, and topographic survey of the area. The survey established that the timber deck of Middle Pier is at about EL 16.7 ft. This is below the target elevation of 12.7 ft NAVD88 (equal to 18.4 feet MLLW) selected for the Public Landing structures for the new construction to consider sea level rise. This will require the deck to be raised by 1.7 feet. Such an action will require demolition of the existing timber portion of the structure and raising the grades of the paved portion. The park area has a general slope upward to the north. Thus the northern portion existing surface will receive new surface treatments, such as paved walkways, as part of the proposed project, but it is generally above the design grade now.

Proceeding southward from that higher area is the structural portion of Middle Pier. On the east side the pier is a continuation of the rip rap sloped fill. On the west side the structure is a granite block seawall that is similar to the other seawalls of the Harbor Park area, with vertical face, a timber deck structure bearing on it and extending over the water. The east side sloping revetment wraps around the south side and ends against the back side of the west side granite wall. The timber deck extends along the westward and southward sides of the seawall, supported on the wall and on a row of timber bearing piles and pile caps. A concrete abutment supports the edge of the wood deck on the south side for the portion with the rip rap slope.

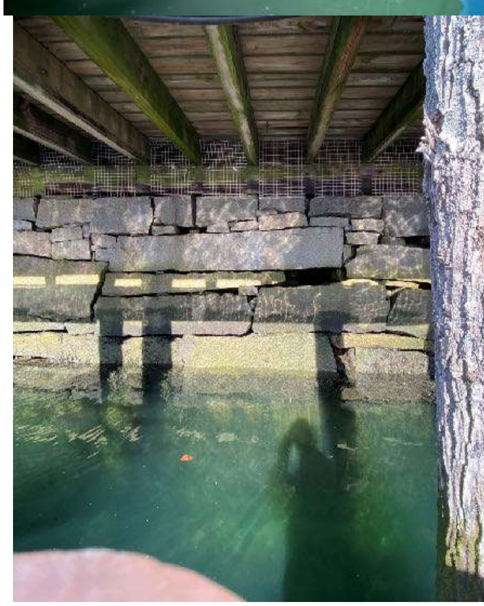
Based on a visual inspection it appears that the pier is in stable and usable condition, but several items are in a poor state. These are discussed below.

<u>Granite Masonry</u>

The seawall is the dry-laid quarried stone typical of coastal New England. Granite wharves of the same construction are generally the most long-lived, but are no longer commonly built. The granite itself will last thousands of years and the simple dry laid stone does not deteriorate with weathering and sea water exposure. The wall appears to be very thick, though it could not be measured. Usually, the base width of these walls will be 1/2 to 2/3 of the height, tapering to one stone wide at the top. It could easily be up to 15 to 20 ft wide at the base.



The timber deck level is about 16.7 ft on the MLLW datum. The wall exposed is approximately 10 ft high on the north end and increasing to 24 ft high above the mudline on the southern face.
The wharf masonry that is visible generally looks good. The main west side wall face is fairly straight and planar. There is a great deal of workmanship required to fit rough stones and this wharf appears to be average for the quality of the stone fit and workmanship, based on the visible face stone.



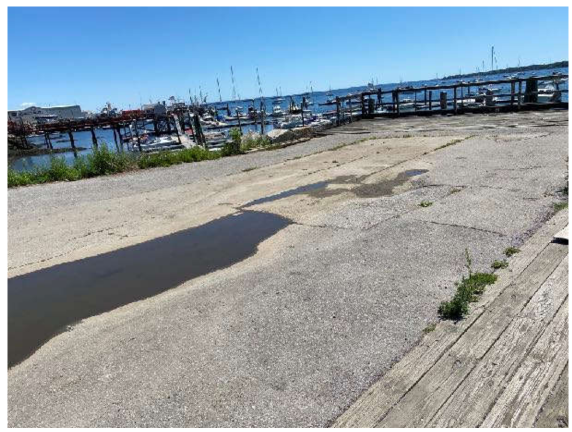
There are a few places where it appears some of the face chinking stones in the horizontal face joints are missing, though we don't know if they were ever there. Because of the irregular heights of the stones, the horizontal joints sometimes need small granite blocks to level and hold the stone above the joint in position. It should be possible to insert new stone, fitted tightly enough to stay. Concrete could be placed as a less desirable alternative.

At the southwest corner, the pier timber deck is supported on a large cantilevered stone. When rebuilt, this area needs attention to reconfigure the timber deck for support further into the main mass of the wall.

The top row of stones is a substantial row of uniform blocks, from what can be seen from below on the floats, and they support the timber deck. This detail will need to be reconfigured when the deck is raised to the higher level.

Fill

A trouble spot with many filled granite piers, is settlement of the top surface and sinkholes forming in the surface. The cause is that the fine soil materials behind the granite wall, which forms the interior of the pier, is washed out by the cycles of water flowing through the wall, due to tides and storms. The wide joints between stones allow a significant flow of seawater in and out of the structure and the fill material. This flow comes in with a rising tide and out with the falling tide, picks up small particles and grains of soil and stone and carries them out when the tide drops, and water flows out through the cracks. Over the years of the constant flow, the finer soil is slowly removed and a void is formed that will eventually collapse and cause internal settlement and/or a sinkhole will appear on the top surface (see picture).



One would think that the erosion would eventually reach an equilibrium where no more small particles are left, so the settlement and sinkholes should happen more in the early years and gradually taper off. In fact, it seems to continue, punctuated when a storm event happens that brings higher water levels to erode newer zones in the fill above that which is normally reached by the tide itself, or causes increased hydraulic pressure in the lower zones to dislodge new material. Even an older wharf like this one continues to see these erosion events, no doubt dependent on the particle size and quality of the fill used in the original construction.

Timber Deck

The timber 10 x 10 pile caps supporting the deck are creosote treated. The deck support joists are 4x10 creosote treated at 24" spacing. The timber deck planks and curbs appear to be creosote also, but are heavily weathered and some may be CCA treated (to be checked).

The West side deck leg is 10 feet wide and 246 ft long, and the south side deck leg is 16 ft wide by 38 ft long. The construction is the same on both legs with 3" nominal deck planking and 6x8 and 8x8 curbs. There was some 4" decking, as well, in

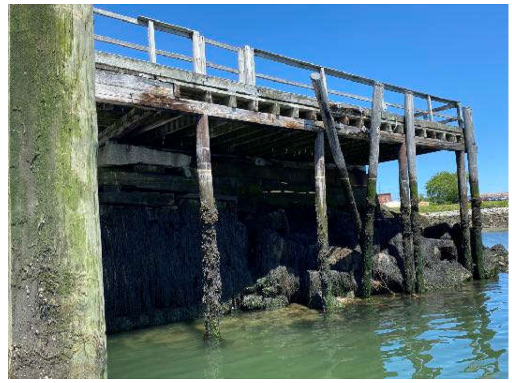
places. The outer edge has handrails attached to the inside face of the curbs (see picture).

There is a general weathered and shabby appearance, particularly the handrails, which are in very poor repair. Many places where the joists are visible, there are splits from the large diameter spikes used to fasten the deck planks, particularly near the ends (see picture below). The splitting is significant to the point that it has released the grip on the spikes holding the planks, so some of the planking has become detached and lifted at one end and creates a tripping hazard.

Before the ban on creosote, it was a very effective preservative against decay and marine borers. The creosote treated piles seem to last a very long time in Maine waters. New creosote-treated piles have not been allowed in new water structures for several decades in Maine and many other states. Most creosote comes as a byproduct from coal tar and the primary market for creosote is currently the treating of timber for railroad ties and utility poles. The EPA has delisted it for most uses other than as a wood preservative because it is carcinogenic. There is quite a lot of information available on creosote regarding a debate over the harmful characteristics and the alternatives to using it. There is some debate on whether there is significant harm to the marine environment when used in marine piling, but it seems to be generally agreed that it should not be used where it can come in contact with humans.

Depending upon local regulations, the creosote treated piles and timber removed will be considered special waste or hazardous waste and must be removed and disposed of as hazardous. The disposal cost could be a substantial cost or not, depending upon the locality and the ability of solid waste disposal, and should be included in the cost estimates.

<u>Piles</u>
Most of the piles on this pier are fender piles which support the float system. In a row behind the fender piles are the bearing piles which support the pier deck.



Fender Piles

The fender piles appear to be fairly new untreated oak piles on the west side. We saw them being installed on one visit last year.

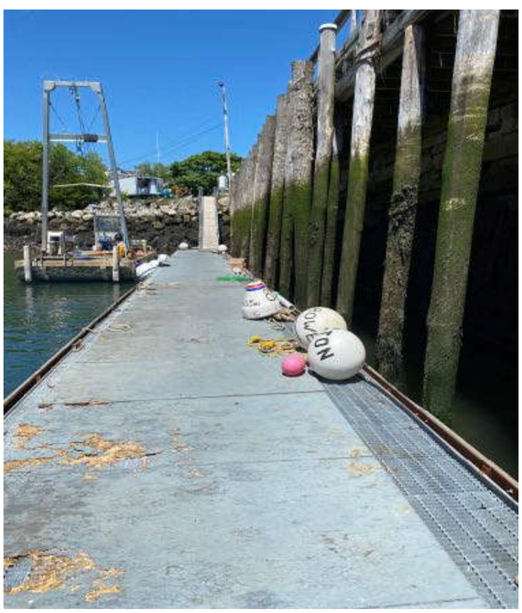
Several older broken fender piles are located on the south side. The piles are fastened to the pile cap. One of the piles is broken free at the top and another is dangling from one bolt (see picture).

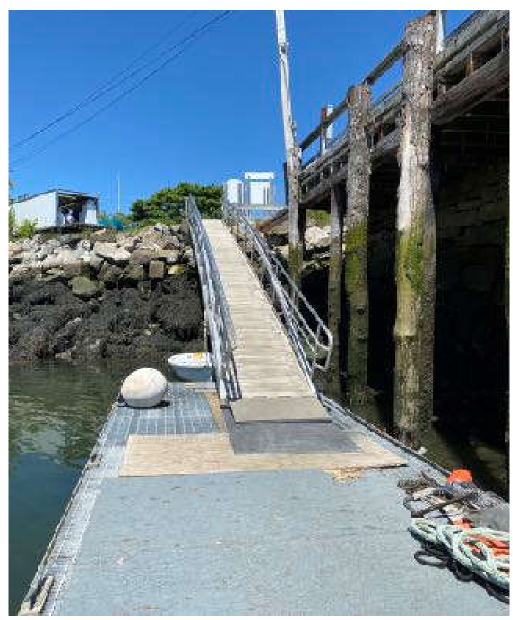
Bearing Piles

The row of bearing piles that supports the outer edge of the timber deck along the south and west sides appears to be creosote treated piles.

Gangway

The existing aluminum gangway appears to be fairly new and meets ADA standards except for length (see picture). The existing abutment is lower than the anticipated deck level for sea level rise, but could perhaps be designed with a short on-grade ramp or steps when the grades are changed in that area. Or it may be easier to rebuild the abutment as part of the pier reconstruction.





Floats

These floats are recycled, obsolete aquaculture cage components used in the Downeast area for Atlantic Salmon operations some 30 years ago. They seem to be satisfactory for their purpose at Middle Pier. They consist of galvanized steel frames with welded steel grating with serrated bars, supported by plastic float drums. The top surface has been covered with painted Oriented Strand Board (OSB) panels, that needs to be renewed. Pressure treated plywood would be preferable to OSB. The outer edges of the original fish cages did not have rub strips or fendering so plain steel bars were welded to the edges. These are beat up and corroded and should be replaced. The steel edges need to be protected since they are causing chafe to the guide piles.

These floats could be reused in the redevelopment of the pier. They should be removed from the water and reconditioned as necessary. This would include inspection and repairs of the metal grating, hinges, edge fendering and flotation. Replacement float drums may be needed if the existing floats are found to be punctured or damaged.

As stated in the Wood Environmental report, the travel range in the vertical direction from tides, is limited by the float collars on the fender piles. They will stop travel at approximately El 14.7 ft, which is several feet shorter than the design level for sea level rise. If a float cannot freely travel upward, significant forces will be exerted on the fasteners and hardware and damage will occur. Additional travel should be accommodated in the design of the replacement structure of the deck.

Electrical

There are two electrical poles on the pier that support lighting and are braced by welded steel brackets at deck level and guy wires. The new project design should eliminate the guy wires. The electrical service panel and wiring on the dock should be designed by a qualified electrical engineer, and would need to be completely replaced in the proposed higher pier construction.

3. DESIGN RECOMMENDATIONS

3.1 General Material Recommendation and Design Basis

COORDINATION OF DESIGN ELEMENTS

The overall project consists of several distinct new and replacement structures and connecting elements such as walkways, ramps, bridges, gangways, railings, and also existing elements that will remain. It will be desirable to define the design aesthetic, styles, materials, and theme throughout the various elements and to what extent these should be related.

Prior to detail design of the particular elements, it will behoove the City to provide the designers with instructions on these elements, particularly the materials, styles and colors that should be used. For example, all of these structures will have guardrails and handrails and it would seem to be desirable to have them consistent. Another example is the walkway surfaces for the Harbor Walk. This will likely require outreach, study, and discussion.

Code requirements for outdoor structure, such as the marine structures in this study, are not well covered in the International Building Code (IBC), which mostly is concerned with buildings and their exterior features. The ADA Guidelines govern public routes and accessibility features and will be incorporated in the design. OSHA has some requirements related to workers and employees which will apply to some features. The City Code Enforcement expertise should be sought early in the process to define the public safety features and requirements desired by the City for this project, overall and in particular, for those areas not covered by other codes.

STRUCTURAL DESIGN REQUIREMENTS

Vehicle and cargo Loads - Decks of Piers shall be designed for a minimum uniform of 200 pounds per square foot (psf) structural live load, or the wheel loads for AASHTO H20-44 Vehicle loading, positioned in any practical travel path on the deck.

An emergency vehicle loading shall be checked, which may exceed otherwise applicable vehicle weight limits. These vehicles can be very heavy, so we will request input from the Rockland Fire Chief on what weight should be used, particularly for fire vehicles.

The maximum vehicle rating for each pier should be posted with signage on the pier and we will advise on the wording to be used.

Pedestrian bridges and gangways shall be designed for a minimum uniform of 85 pounds per square foot (psf) structural live load with a deflection under a combination of dead load and live load not to exceed 1/180 of the span. Under uniform load of 50 pounds per square foot (psf) structural live load, the deflection shall not exceed 1/360 of the span. A concentrated live load of 500 lbs shall be applied anywhere on the walking surface.

Wind Load - wind loading of 35 lbs/sq.ft on the full projected area of the exterior, as if enclosed, applied in any horizontal direction.

Handrails shall be designed to be capable of resisting a 200-pound concentrated load, or 50 pounds per lineal foot uniform load, in any direction.

TIMBER CONSTRUCTION

TIMBER PILES
Timber piles shall consist of a round, clean-peeled, single piece from butt to tip, complying with ASTM D 25. The timber will be free from decay, unsound knots, knots in clusters or groups, checking, and excessive bends.

Pile Species:
Float Piles shall be greenheart, peeled, and untreated. Where pile indicates potential for splitting after drying, bind butt ends with a stainless-steel strap to restrain splitting.

Pier structural piles and others where specified as Southern Pine and shall be treated with CCA to 2.50 pcf. The tip diameter should be 8" minimum, with the butt diameter measured 3 ft from end (12" minimum). Lengths will not be determined in this study and should be determined with test piles before ordering or based on records of the City prior pile installations in the same area.

Float piles covers shall consist of black UV-resistant polyethylene caps, flat style.

Pile installation will use pile driving equipment of type generally used in standard pile driving practice. It is anticipated that the piles will be installed by vibratory pile driver to refusal, then proofed with an impact hammer to ensure pile is on bedrock.

TIMBER FRAMING
Timber shall be supplied with documentation showing that it has been obtained using certified sustainable forest practices by one of the certification agencies.

The timber species shall be pressure treated, Southern Pine.

Timber grade should be No. 1 for exposed lumber decking and handrails and No. 2 Grade minimum or as indicated on drawings where a better grade is required.

The timber preservative treatment for piles and bracing, pile caps and deck members shall be treated for Saltwater immersion or splash. They shall be pressure treated with CCA (Chromated Copper Arsenate) to AWPA Use Category 5A except any timber used with any portion below extreme high water shall have a minimum retention of 2.5 pcf.

Members located above the water line and where contact with users generally occurs, shall be protected with a waterborne copper-based preservative suitable for AWPA Use Category 4B. Acceptable chemicals include Micronized Copper Azole (µCA) – 0.23 pcf and Micronized Copper Quat (MCQ)- 0.60 pcf. The preservative industry is in a flux in this regard and many chemicals have been promoted and then abandoned later, so it is generally left open to multiple choices of products for members above the water line.

Timber is to be surfaced on 4 sides where it is exposed at deck level or above, such as for decking, curbs, chocks, handrails, and ladders. Timber shall be dressed on 2 sides where the critical dimension is in only one direction, such as for pile caps and stringers, and can be rough (undressed) on all sides where dimensional control is not required, such as for bracing.

The walkway deck surfaces shall be minimum of 5/4” x 6” Southern Pine decking, pressure treated. Heavier timber planking or concrete will be used where vehicles will/may be operating.

HARDWARE, CONNECTORS, ANCHORS, ACCESSORIES

Provide fabricated structural steel (ASTM A 36) shapes, plates and bars, welded into assemblies of types and sizes indicated, with steel bolts (ASTM A 307), lag bolts, and other fasteners as required.

Each assembly and fastener unit shall be finished with hot-dip zinc coating (ASTM A 153) and galvanized after assembly.

Decking fasteners shall be stainless steel deck screws or equivalent, as approved by Engineer for saltwater exposure and suitable for the preservative in the lumber.

Epoxy coated rebar or galvanized threaded rod or carriage bolts shall be used for draft pins.

ASTM A307, hot-dipped galvanized bolts shall be used. Bolts below HAT elevation shall be minimum 1” diameter.

Galvanized dock washers, ogee washers, or plate washers shall be used with a minimum 1/4" thickness. They should be hot-dipped galvanized.

<u>CONCRETE STRUCTURES</u>

CONCRETE

Codes and Standards: Comply with MaineDOT Standard Specifications

ACI 301 -	Specifications for Structural Concrete for Buildings
ACI 305 -	Hot Weather Concreting
ACI 306 -	Cold Weather Concreting
ACI 318 -	Building Code Requirements for Reinforced Concrete
ACI 347 -	Recommended Practice for Concrete Form work

Concrete Reinforcing Steel Institute (CRSI), "Manual of Standard Practice"

CONCRETE MATERIALS

Precast and Cast in Place Concrete shall be 5000 psi 28-day compressive strength: W/C ratio, 0.40 maximum (air-entrained).

Portland Cement: ASTM C 150, Type II, or Type I with a pozzolan and corrosion inhibitor. Type III may only be used if tricalcium aluminate content (C3A) is between 4% and 10%. This improves corrosion resistance for the reinforcement.

Normal Weight Aggregates: ASTM C 33. Provide aggregates from a single source for all exposed concrete. Request evidence of good performance of the aggregate from this source or provide testing results for potential for alkali silica reactivity, by ASTM Guide C 295, Test Methods C 227 or C 289.

Fly ash - ASTM C 618 Class F or ASTM C1240. Use of pozzolans to replace 20 to 30% of the cement is encouraged to reduce susceptibility to Alkali Silica Reactivity and to decrease permeability and is more sustainable since it reduces the amount of cement used.

Air-Entraining Admixture: ASTM C260, certified by manufacturer to be compatible with other required admixtures. Increases freeze-thaw resistance of concrete.

Corrosion Inhibiting Admixture: “DCI “; W.R Grace shall be provided in all waterfront concrete. Provide at manufacturer’s recommended dosage, minimum 4 gallons per CY.

Reinforcing Bars: ASTM A 615, Grade 60, deformed, epoxy coated.

Supports for Reinforcement: Bolsters, chairs, spacers, and other devices for spacing, supporting, and fastening reinforcing bars and welded wire fabric in place. Use wire bar type supports complying with CRSI specifications, made from stainless steel.

ALUMINUM STRUCTURES

Aluminum gangways and aluminum walkway spans shall be complete and shall include all bolts, nuts, washers, hinges and connection hardware, rollers, transition plates, lifting harness, wear plates on floats, and miscellaneous hardware required to construct and connect to structures. Gangways shall incorporate lifting points and a wire rope sling for raising to a horizontal storage position. Lifting point eye bolt shall be provided in an overhead beam for Owner-supplied chain hoist. Provision shall be included for raising and storing those gangways during winter months in a raised position.

New Gangways shall be 80’-0” long measured from the top hinge pin to the end of the deck surface at the float, not including the transition plates.

The clear width between handrails shall be 4’-0” for gangways and 6’-0 for walkways

The gangways and walkways shall meet the requirements for handrail height, clear width, length, slope, and deck surface treatment, as required to meet the Americans with Disabilities Act (ADA). The top of the side truss on each side shall be 42” and a separate handrail shall be provided at a height between 34” to 38”, above the gangway deck surface.

One intermediate horizontal rail and a toe rail shall be provided along each side. Additional protection, such as balusters or mesh, should be determined by the CEO for the City.

The gangway and walkway deck surface shall be continuous, without tripping hazards or gaps in excess of ADA requirements. A hinged transition checker plate is required at the upper end between the end the dock and at the lower end and the float, for all gangways

The gangway walking surface shall consist of an aluminum (with non-slip) top surface and a McNichols Plank Grating Traction Tread. Planks shall be welded to the supports and adjacent planks shall be stitch welded.

Gangway rollers shall be non-marring of the dock surface and rated to carry the gangway dead load plus live load as may be applied to the wheels or rollers. Axles and hardware shall be Type 316 stainless steel. Rollers shall be UHMW polyethylene.

EDGE PROTECTION - GENERAL FOR ALL STRUCTURES EXCEPT GANGWAYS

Pedestrian guards shall be required to protect edges with a drop exceeding 30 inches as defined in the IBC Code. Guardrails shall be 42” high, measured above the walking surface.

On ramps, handrails shall be located parallel to the ramp surface at a height of 34" to 38" measured to the top of the gripping surface. Generally, this will require separate handrails mounted on the side of the guardrail.

Gripping surfaces shall be continuous. Handrails shall not rotate within their fittings. Ends shall be either rounded or returned smoothly to the floor, or end post and extend (as shown on the plans) at least 12" beyond the top and bottom of the ramp walking surface. Handrails shall be made from aluminum pipe with a minimum 1 1/4" nominal pipe size. The clear space between the handrail and other structural elements behind the handrail shall be a minimum of 1½".

The loading sides of piers do not require guards but shall have 10” high curbs for vehicles.

3.2 Public Landing

Design Recommendations

- Replace expand pier* with timber piles and concrete or wood decking
- Reuse the existing 80 ft aluminum gangway and add as second similar gangway*
- Reconfigure floats and re-use existing floats, as much as possible*
- Consider larger wave attenuation floats to protect inner boat basin and serve larger vessels*
- Establish float removal procedure and design float structure accordingly
- Relocate piling for floats new arrangement and provide comprehensive rehabilitation of the float/piling interface issues

Resiliency and Sustainability

- Raise pier to increase resiliency with target elevation of 12.7 (NAVD88) / 18.4 (MLLW)*
- Consider pile connections that allow raising deck height in the future…"flexible resiliency"
- Consider panelized deck structure, with lifting points, for ease of future removal
- Choose sustainable pile material that considers useful life of deck and future resiliency
- Consider requirement of third-party certified sustainable products with Chain of Custody documentation
- Choose deck and rail materials that consider maintenance costs and sustainability
- Concrete will incorporate recycled fly ash to reduce cement requirements and its Co2 footprint

*In accordance with Downtown Waterfront Concept

3.3 Harbor Park Seawall

Design Recommendations

- Raise seawall to increase resiliency with target elevation of 12.7 (NAVD88) / 18.4 (MLLW)*
- Combination of pile-supported and at-grade harbor walk
- Incorporate granite blocks to raise wall to limit size of concrete cap

Resiliency and Sustainability

- Consider pile connections that allow raising deck height in the future…"flexible resiliency"
- Incorporate granite where possible or sustainability
- Choose deck and rail materials that consider maintenance costs and sustainability
- Concrete will incorporate recycled fly ash to reduce cement requirements and its Co2 footprint

*In accordance with Downtown Waterfront Concept

3.4 Harbor Park Connector

Design Recommendations

- Pile supported fixed pier connector with bridge
- Bridge span to be approximately 80' that represents an aesthetic opportunity
- Plan for ramp and stair connection to existing Pearl pier
- Construct pier with target elevation of 12.7 (NAVD88) / 18.4 (MLLW)*

Resiliency and Sustainability

- Consider pile connections that allow raising deck/bridge height in the future…"flexible resiliency"
- Choose deck and rail materials that consider maintenance costs and sustainability

3.5 Middle Pier

Design Recommendations

- Replace expand pier* with timber piles and concrete or wood decking*
- Expand floats and re-use existing floats as much as possible*
- Construct pier with target elevation of 12.7 (NAVD88) / 18.4 (MLLW)*
- Construct proposed dinghy docks with minimum of -2' depth to maximize use
- Establish float removal procedure and design float structure accordingly

*In accordance with Downtown Waterfront Concept

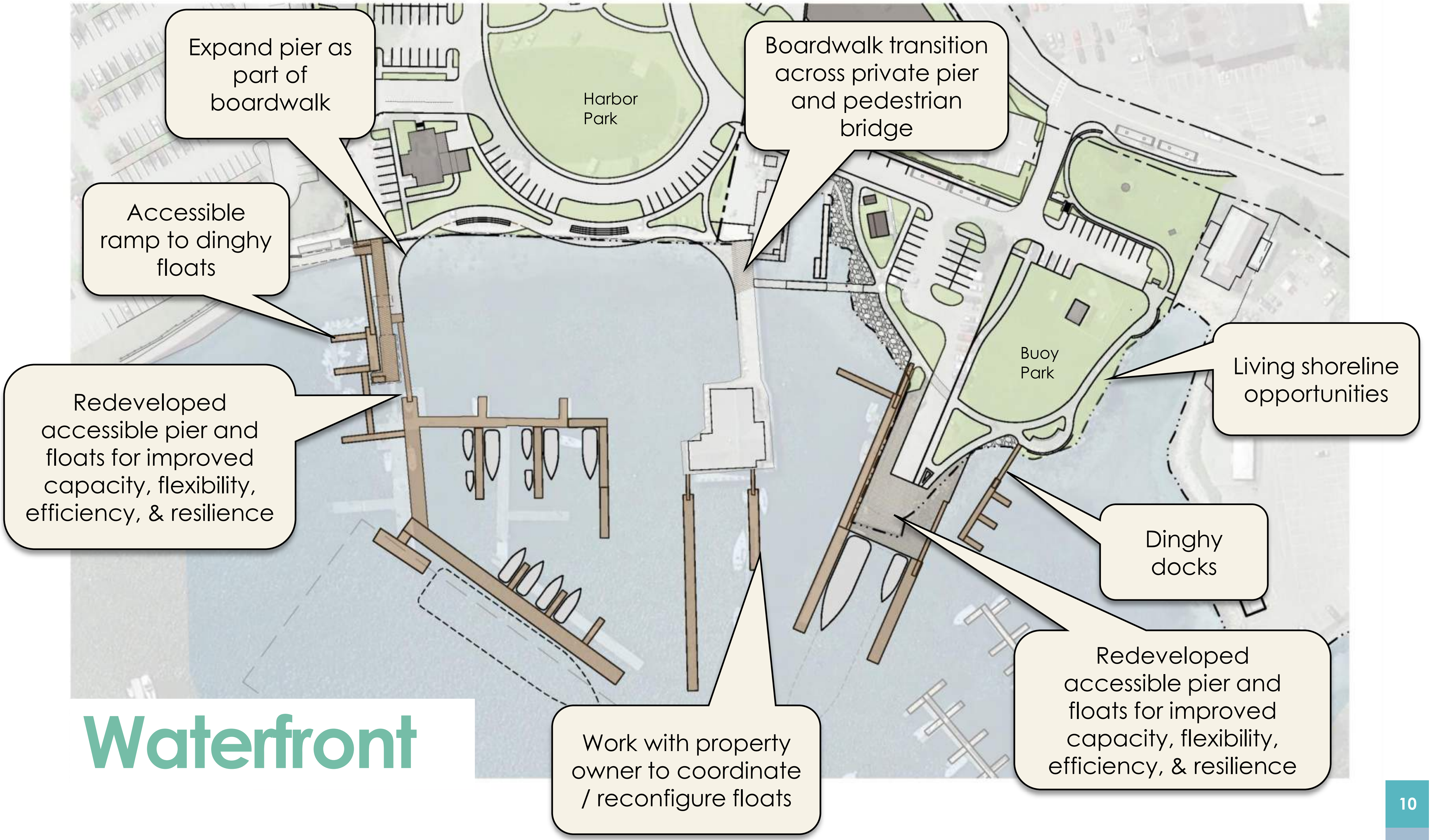
Resiliency and Sustainability

- Consider pile connections that allow raising deck/bridge height in the future…"flexible resiliency"
- Choose deck and rail materials that consider maintenance costs and sustainability

APPENDIX A

DOWNTOWN WATERFRONT CONCEPT PLAN

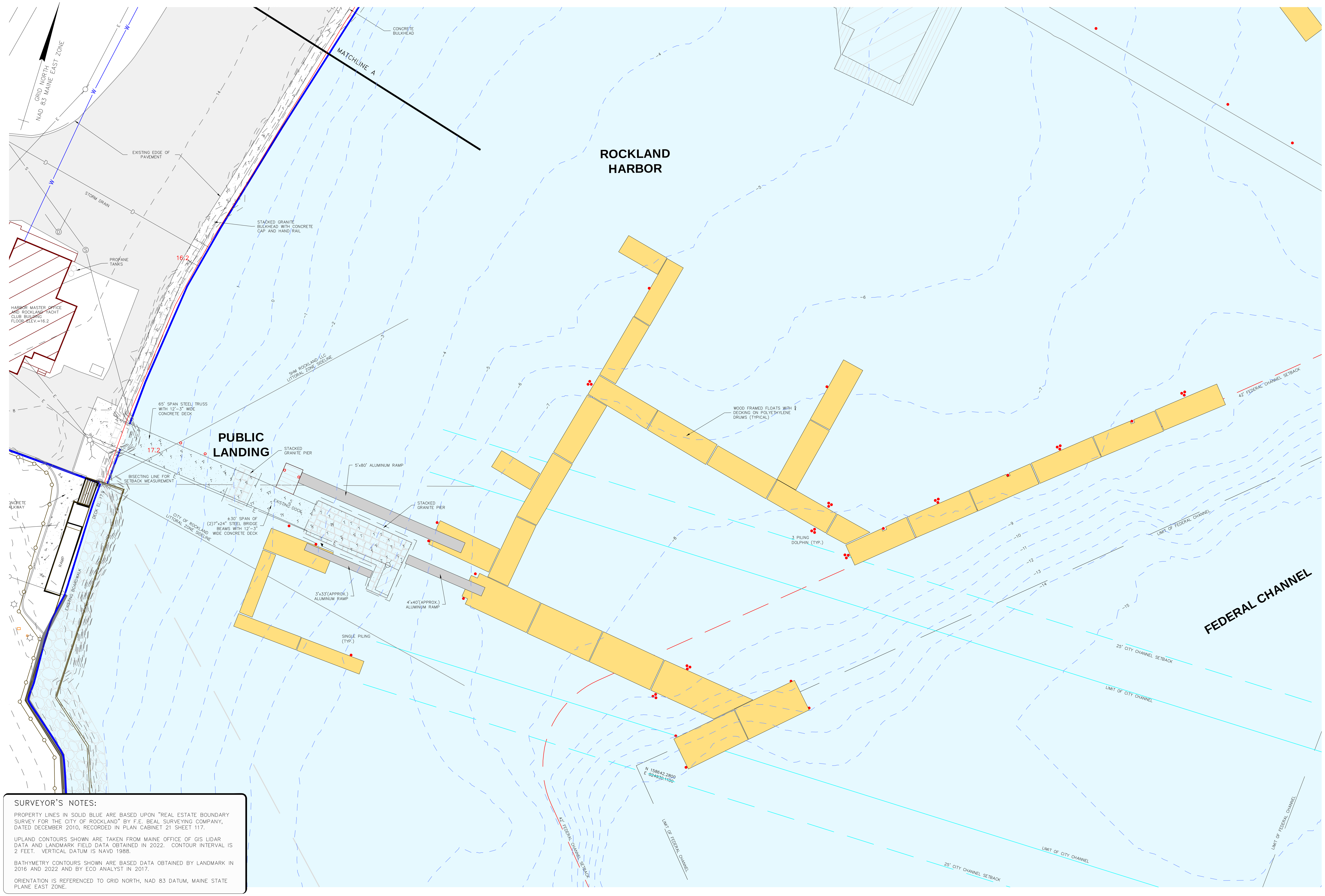
Concept Plan


Expand pier as part of boardwalk
Harbor Park
Boardwalk transition across private pier and pedestrian bridge
Accessible ramp to dinghy floats
Buoy Park
Living shoreline opportunities
Redeveloped accessible pier and floats for improved capacity, flexibility, efficiency, & resilience
Dinghy docks
Work with property owner to coordinate / reconfigure floats
Redeveloped accessible pier and floats for improved capacity, flexibility, efficiency, & resilience

Waterfront

APPENDIX B

EXISTING CONDITIONS PLANS



LEGEND:

- IRON PIN SET OR FOUND BY BEAL
- UTILITY POLE
- WATER VALVE
- HYDRANT
- SEWER MANHOLE
- CATCH BASIN
- DECIDUOUS TREE
- EXISTING FENCE
- EXISTING DRAINAGE
- EXISTING OVERHEAD UTILITY
- EXISTING WATER MAIN
- EXISTING SEWER MAIN
- EXISTING WATER VALVE
- EXISTING HYDRANT
- EXISTING SEWER MANHOLE
- EXISTING CATCHBASIN
- 10.5 EXISTING SPOT ELEVATION
- 14 EXISTING CONTOUR (LIDAR)
- 10 BATHYMETRY (2017)
- BOUNDARY LINE (BEAL)
- TAX MAP LINE
- EXISTING FLOAT PILE
- EXISTING SUPPORT PILE

SURVEYOR'S NOTES:

PROPERTY LINES IN SOLID BLUE ARE BASED UPON "REAL ESTATE BOUNDARY SURVEY FOR THE CITY OF ROCKLAND" BY F.E. BEAL SURVEYING COMPANY, DATED DECEMBER 2010, RECORDED IN PLAN CABINET 21 SHEET 117.

UPLAND CONTOURS SHOWN ARE TAKEN FROM MAINE OFFICE OF GIS LIDAR DATA AND LANDMARK FIELD DATA OBTAINED IN 2022. CONTOUR INTERVAL IS 2 FEET. VERTICAL DATUM IS NAVD 1988.

BATHYMETRY CONTOURS SHOWN ARE BASED DATA OBTAINED BY LANDMARK IN 2016 AND 2022 AND BY ECO ANALYST IN 2017.

ORIENTATION IS REFERENCED TO GRID NORTH, NAD 83 DATUM, MAINE STATE PLANE EAST ZONE.

LANDMARK CORPORATION
SURVEYORS & ENGINEERS
135 ROCKLAND STREET ROCKPORT, MAINE 04856 PHONE: (207) 236-6757 WWW.LANDMARKMAINE.COM

FIELD WORK DATE:
FIELD WORK BY: KMB/EST
DRAFTED BY: KBM/MJS
CHECKED BY: MJS
PLAN DATE: NOVEMBER 1, 2022

GRAPHIC SCALE
20 0 10 20 40
1 inch = 20 ft.

EXISTING CONDITIONS PLAN

STATE OF MAINE
MICHAEL J. SABATINI
9053
LICENSED
PROFESSIONAL ENGINEER
11/1/22

CITY OF ROCKLAND

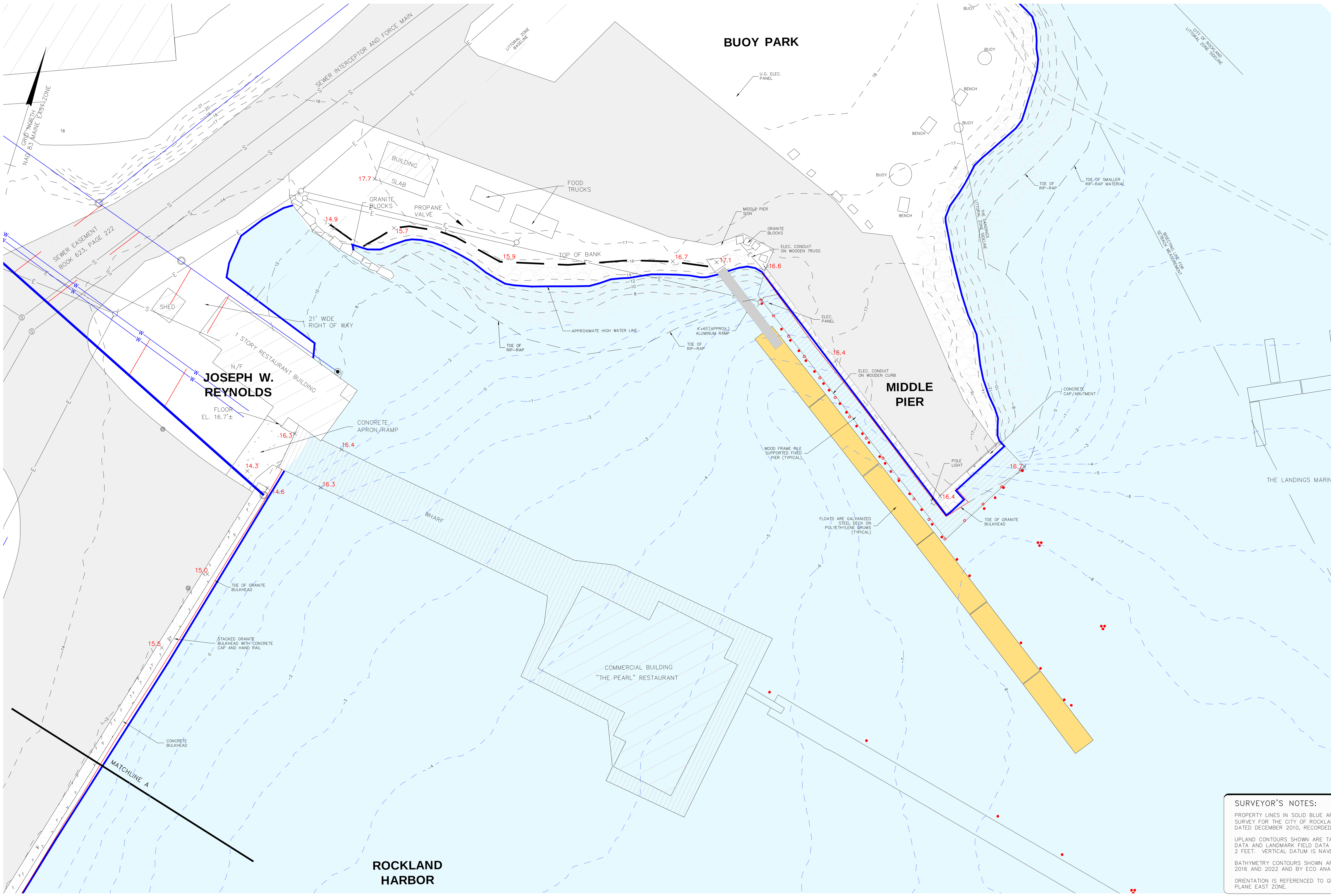
DOWNTOWN WATERFRONT MARINE INFRASTRUCTURE
AT HARBOR AND BUOY PARKS

ROCKLAND, MAINE

KNOX COUNTY

SCALE: 1" = 30' JOB No.: 16 - 012 SHEET 1 OF 2

LANDMARK CORPORATION



LEGEND:

- IRON PIN SET OR FOUND BY BEAL
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- WV WATER VALVE
- HYDRANT
- SEWER MANHOLE
- CATCH BASIN
- DECIDUOUS TREE
- EXISTING FENCE
- D EXISTING DRAINAGE
- E EXISTING OVERHEAD UTILITY
- W EXISTING WATER MAIN
- S EXISTING SEWER MAIN
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- EXISTING SUPPORT PILE

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FIELD WORK BY:	KMB/EST
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GRAPHIC SCALE

20 0 10 20 40

1 inch = 20 ft.

EXISTING CONDITIONS PLAN

STATE OF MAINE
MICHAEL J. SABATINI
9053
LICENSED
PROFESSIONAL ENGINEER
11/1/22

CITY OF ROCKLAND

HARBOR AND BUOY PARK MARINE INFRASTRUCTURE

ROCKLAND, MAINE

KNOX COUNTY

SCALE: 1" = 30'	JOB No.: 16 - 012	SHEET 2 OF 2

LANDMARK CORPORATION

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APPENDIX C

VULNERABILITY ASSESSMENT AND RESILIENCE PLANNING, MIDDLE PIER
WOOD ENVIRONMENT & INFRASTRUCTURE SOLUTIONS, INC.

Wood Environment & Infrastructure Solutions, Inc.
511 Congress Street, Suite 200
Portland, ME 04101, USA
T: +1 (207) 775-5401
www.woodplc.com

20 December 2019

Project Number: 3611191238

Kathleen Leyden
Director, Maine Coastal Program
32 Blossom Lane
Augusta, ME. 04333-0021

Subject: Vulnerability Assessment and Resilience Planning, Middle Pier, Rockland, Maine
Penobscot Bay Working Waterfront Resiliency Analysis
State of Maine, Department of Marine Resources

Wood Environment & Infrastructure Solutions, Inc. (Wood) is pleased to provide the Maine Department of Marine Resources (DMR) this report on the baseline characterization, vulnerability assessment and resilience planning for the Middle Pier in Rockland, Maine. This report provides findings for one of ten sites included in DMR's Penobscot Bay Working Waterfront Resiliency Analysis project. Reports on the other ten sites are provided under separate cover. Our work was performed in general accordance with the scope of work and the terms and conditions included in Wood's proposal dated 1 March 2019.

1.0 INTRODUCTION

As proposed for DMR's Penobscot Bay Working Waterfront Resiliency Analysis Project, Wood conducted an assessment of the Middle Pier in Rockland, Maine which included:

- Facility baseline characterization including a review of available site documents, interviews with community representatives, survey of site topography and elevations of key site features, and review of the general condition of existing site structures by a Wood structural engineer;
- Facility vulnerability analyses based on the baseline survey data, condition of structures, and modelling of potential storm surge and wave affects under three sea-level rise scenarios; and
- Development of resilience measures, including strategies for incremental adaptation under the modelled sea level rise scenarios.

This report contains a summary of our document review, personnel interviews, structural observations, photographs documenting our observations (**Appendix A**), and the approximate location of potential structural deficiencies. Following our analysis of the site and as part of the vulnerability analysis, we were able to identify the risks for the affected site features (see **Table 5**) from inundation data. Inundation maps developed for the site by Wood's consulting partner, Woods Hole Group (WHG) are provided in **Appendix B**. The vulnerability analysis establishes the future risk framework for the site and its structural features. Wood has evaluated the degree of impact of these site-specific vulnerabilities, and we have provided recommendations for improved resilience (e.g., repair, reinforcement) in relation to the feature's immediate performance and/or expected performance per the vulnerability analysis.

As part of the subsequent discussion, the following terms are defined below:

Base Flood Elevation (BFE) -	Elevation of flooding, including wave height, having a 1% chance of being equaled or exceeded in any given year.
Checks	A separation of the wood occurring across or through the rings of annual growth and usually as a result of seasoning.
Coastal High hazard Area (CHHA) -	Area within a special flood hazard area extending from off-shore to the inland limit of a primary frontal dune along an open coast and any other area that is subject to high velocity wave action.
Design Flood Elevation (DFE)	Based on the design flood, the DFE is the higher of the base flood elevation (BFE) shown on FIRMs prepared by FEMA or the flood elevations shown on the map adopted by a community.
FIRM -	Flood Insurance Rate Map. Official map of a community on which FEMA has delineated both special flood hazard areas and the risk premium zones applicable to the community.
Highest Annual Tide (HAT) –	The elevation of the highest predicted astronomical tide expected to occur at a specific tide station over the National Tidal Datum Epoch.
Mean Higher High Water (MHHW) –	The average of the higher high water height of each tidal day observed over the National Tidal Datum Epoch. The highest high tide or water height is referred to as the Highest Astronomical Tide (HAT) and is defined as the highest level which can be predicted to occur under average meteorological conditions and any combination of astronomical conditions.
National Tidal Datum Epoch –	The specific 19-year period (Currently 1983 to 2001) adopted by the National Ocean Service as the official time segment over which tide observations are taken and reduced to obtain mean values (Mean Lower Low Water, etc.) for tidal datums.
Pre-FIRM	Construction or substantial improvement occurred on or before December 31, 1974.
Shakes	Lengthwise separations of the wood along the grain, usually occurring between or through the rings of annual growth.
Splits	A separation of the wood through the piece to the opposite surface or to an adjoining surface due to tearing apart of the wood cells.
Still Water Elevation –	Elevation that the surface of the water would assume in the absence of waves referenced to a specified vertical datum at the defined recurrence interval.
Wave Height –	Vertical distance between the crest and the trough of a wave.

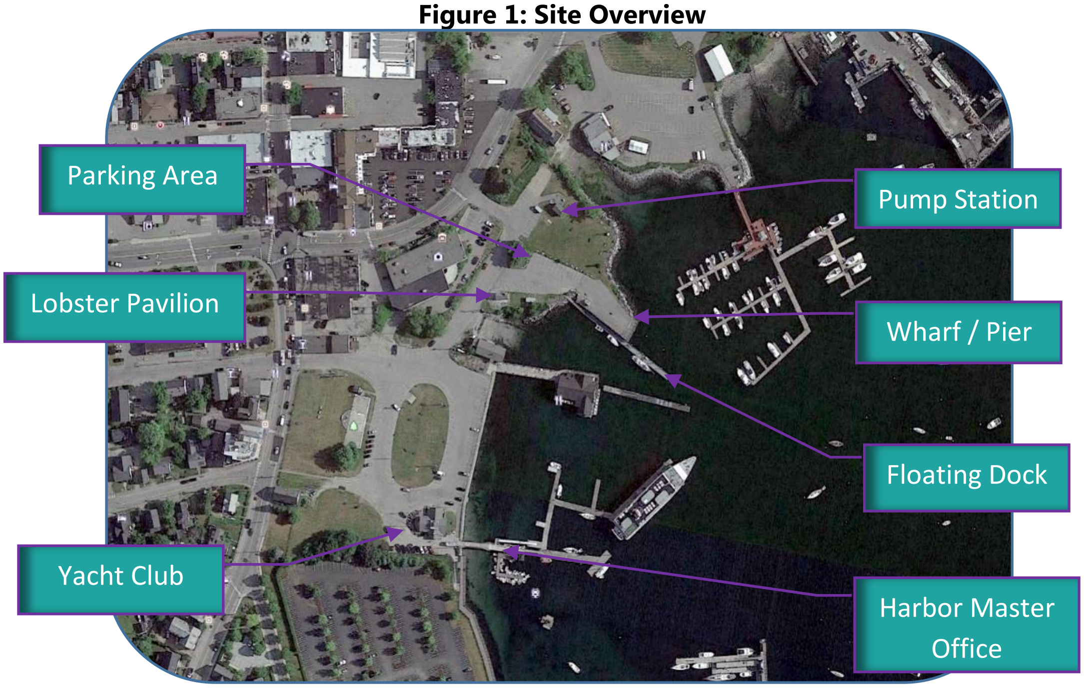
2.0 DOCUMENT REVIEW AND PERSONNEL INTERVIEWS

Wood was escorted by Mr. Matt Ripley, the Harbor Master, during a site visit on 19 June 2019. We discussed the site features and historical development of the site. Harbor Master Ripley indicated that the Patriots Day storm of April 2007 was the last notable storm event which impacted the site. Mr. Ripley also noted the current use of the site for public access and commercial passenger vessels. The Harbor Management Plan was noted as an effort taking place this year and will include discussion of the Middle Pier site, community needs, and plans for future improvements. Otherwise, they have been active in applying for federal grants to address pending capital improvement projects. The following is a summary of key site features identified by Harbor Master Ripley during our discussion:

- The site consists of the wharf, parking area, and a public park/greenspace (See **Figure 1** below).
- Structures located on site include a lobster cooking pavilion and a sewer pump station with a wet well.
- Adjacent to Middle Pier is the main harbour which includes a harbor master office, a yacht club and public restrooms.
- The wharf is constructed of quarried granite blocks, timber piles and timber framing.
- A wooden floating dock is located on the south side of the wharf.
- A breakwater constructed of quarried granite exists roughly 1.5 miles east of the site.
- There is no ongoing maintenance plan in place; maintenance is addressed, as needed, when a deficiency is identified.

Prior to the site visit, Harbor Master Ripley provided a plot plan, parcel map and flood map published by the Federal Emergency Management Agency (FEMA) for the site. No structural plans or as-built drawings were available for our review. Following our site visit we received a copy of the building permit application for an expansion of the Chamber of Commerce/Harbor Master office.

Figure 1: Site Overview



3.0 OBSERVATIONS AND FINDINGS

Tirrell Day and Lane Gray of Wood performed a site assessment and gathered geospatial data for key site features during the 19 June 2019 visit. This assessment included documenting the general condition and recording elevations of key features and structures. At the request of the City of Rockland, two city-owned properties at the Middle Pier were evaluated and are referenced herein as Sites 1 and 2. Photos of the sites and Wood's noteworthy observations are included in the Photolog

(**Appendix A**). The site facilities and their associated elevations can be found in **Table 1** for reference. During our site visit we observed a change in tidal elevation of roughly eight (8) feet, with documented elevations between -3.4 ft and 4.6 ft (predicted min. of -6.3 ft, max. of 5.8 ft). This fluctuation is in line with normal values for this time of year and location. These conditions were taken into consideration during our assessment.

3.1 Property Overview

Site 1

This site is a 2-acre property containing a combination waterfront pier and wharf structure, parking lot, and public park/greenspace (**See Photographs 1 - 10, Appendix A**). The pier and wharf are located at the southeast corner of the site. A floating dock is located on the south side of the wharf for access to the charter boats (**Photograph 11 - 15**). The floating dock gangway is attached to the wharf via anchorage to a concrete footing. The floats are attached to the wharf's exterior timber piles with mooring chains. Wood observed the function of the gangway and floats during tidal action and the system appeared to function as intended. Access to the floating dock is provided at the wharf via the parking lot. The paved area covers roughly half of the site, some of which is access or parking, whereas the remaining portion extends over the wharf.

The wharf appears to be constructed of stacked granite blocks as a substructure with crushed stone fill of large to medium diameter between the granite blocks and the surface above (**Photograph 9 & 10**). Subsurface conditions for the site were not verified by testing, however information provided from the United States Geological Survey (USGS) indicate silty and gravelly sandy loams are typical for this area.

The adjacent pier, at the perimeter of the south and east side of the wharf, is constructed of timber framing on timber piles. Timber framing is attached using a combination of through bolts and large diameter nails, is supported at the extremity by piles and at the wharf interface by either a concrete abutment or granite blocks. The abutment is a concrete structure which appears to be cast on the granite substrate (**Photograph 16**). Attachment of the timber framing to the abutment or granite blocks with an anchor or other fastening mechanism could not be confirmed. Decking is attached to framing via framing nails. Perimeter piles were commonly capped with vinyl covers where exposed from above. Shoreline protection exists beyond the extents of the wharf and pier in each direction and is provided by means of large riprap (**Photographs 24 & 25**). Site utilities include electrical and water, whereas only water is provided at the floating dock (**Photographs 15, 17 & 18**).

Table 1: Site Elevations

Facility	***Lowest Horizontal Member***	***Lowest Deck or Adjacent Grade***	***First Finished Floor***	***Lowest Opening/ Critical Elevation***
Source*	***Estimate (ft)***	***Survey (ft)***	***Estimate (ft)***	***Estimate (ft)***
Pier	9.5	10.5	n/a	n/a
Wharf	9.5	10.2	n/a	n/a
Floating Dock	9.5	11.36	n/a	n/a
Pavilion	n/a	11	11.33	11.33
Pump Station	n/a	14.4	15.07	16.4
Shoreline Protection	n/a	11	n/a	16
Harbor Master Office	9	n/a	9	12
Yacht Club/ Restrooms	10.5	n/a	11	14

**Estimates indicate measurements referenced or derived from the actual site survey data.*

Site structures include a sewage pump station and lobster pavilion for cooking during events. The pump station appears to be a wood-framed structure on stem wall with gable type roof (**Photographs 19 & 20**). The building has four (4) windows, one main access door, and a large rear louver. Exterior cover appears to be vinyl siding and asphalt shingle roofing. What appears to be a concrete slab on grade exists at the north side of the building to provide access to the door and the associated sewer well. The top of grade at the building is roughly 15 feet with a finish floor elevation (FFE) approximately 8 inches above the current grade.

The lobster pavilion (**Photographs 21 – 23**) appears to be a metal-framed building with brick façade on block infill walls, with an asphalt-shingled open gable roof. The structure has no doors or windows but constructed with large openings framed by stem walls on three sides and open on the remaining side. Support for the openings and subsequently the roof framing is provided by steel beams. A large chimney exists at the northeast corner of the roof which appears to be wood-framed. Lighting and other utilities are provided at the building interior.

Site 2

At the request of the City of Rockland, Site 2 was added to the inundation/flood analysis conducted by WHG. Site features observed by Wood include the Harbor Master's Office (**Photograph 28**), a building which houses a yacht club, and public restrooms (**Photograph 29**). The Harbor Master's office is located on a narrow wharf which appears to be constructed of stacked granite blocks. The yacht club building is located adjacent to the paved area, with the restrooms accessible from the west side. The FFE of the building is roughly 2 ft above the top of pavement elevation (approx. elevation 9 feet). The subsurface construction of the site could not be readily observed. A sea wall constructed of stacked granite borders the site at the shoreline. The site is largely paved as it extends up to the access road elevation (approx. elevation 16 ft). A series of gangways and floating docks provide access to boats and the various vessels for mooring (**Photographs 26 & 27**).

3.2 Noted Deficiencies

The wharf structure, particularly the stacked granite foundation, exhibited no apparent signs of translation or dislodgment for the large members. Smaller stone material was noted between the surface and granite blocks, observable through the large openings between the blocks. Above this area on the pavement, large cracks and dips are observed (**Photographs 30 – 32**). Previous repair work was also noted at some locations (**Photographs 31 & 32**).

As earlier mentioned, an abutment is used to support the pier framing at one end. Positive attachment (anchorage or other mechanical fastener) of the concrete abutment to the granite blocks or attachment of the wood framing to the abutment and/or the granite blocks could not be confirmed. The abutment stem and footing appear to be composed of two separately poured elements (**Photograph 16**). Wooden members, which include piles, stringers, and decking exhibit signs of moderate deterioration due to weathering and/or microbial attack. Some minor to moderate conditions of shakes, checks and splits were observed throughout. For those piles observed, signs of infestations such as marine borers were not noted in the tidal zone.

3.3 Risk Framework

As a basis for the vulnerability analysis, water surface elevation exposure profiles under various projected environmental conditions were developed by WHG which summarize current and potential future tidal and storm surge inundation/wave impacts. The key flood elevation profiles provided include the Mean Higher High Water (MHHW), the Highest Astronomical Tide (HAT), the 1% Still Water Level, and the Base Flood Elevation (BFE). Values for these scenarios are site specific and take into consideration the topographic survey data obtained by Wood.

The MHHW and HAT tidal datums (present day) were sourced from the nearest long-term NOAA tide station and from spatial files developed by Maine Geological Survey[1]. The 1%-annual-chance still water level (present day) was obtained from the 2016 FEMA Flood Insurance Study for Knox County.

[1] https://www.maine.gov/dacf/mgs/hazards/highest_tide_line/index.shtml

Table 2: Flood Modelling Data Summary - Site 1

Scenario	*MHHW*	*HAT*	*1% Still Water Level*	*1% Wave Crest Elevation (BFE)*
Present day	4.8	7.1	9.0	11-15
Short Term (+1 ft)	5.8	8.1	10.0	12-17
Mid Term (+2 ft)	6.8	9.1	11.0	13-18
Long Term (+4 ft)	8.8	11.1	13.0	15-20

Table 3: Flood Modelling Data Summary - Site 2

Scenario	**MHHW**	*HAT*	*1% Still Water Level*	*1% Wave Crest Elevation (BFE)*
Present day	4.8	7.1	9.0	11-15
Short Term (+1 ft)	5.8	8.1	10.0	12-17
Mid Term (+2 ft)	6.8	9.1	11.0	13-18
Long Term (+4 ft)	8.8	11.1	13.0	16-20

Site-specific wave modelling was conducted for existing and future sea levels to better quantify wave hazards and potential increases in wave heights at the site. Wave modelling was conducted using FEMA's overland wave modelling approach for consistency in providing an estimate of the 1% BFE for the future scenarios.

For potential future flood impacts, relative sea level rise (SLR) scenarios were reviewed using the U.S. Army Corps of Engineers' Sea-Level Change Curve Calculator (Version 2017.55), specifying the Bar Harbor long-term tide gauge, a regionally-informed vertical land movement rate (from NOAA), and the NOAA et. al (2017)[2] SLR curves.

In discussion with the project team, the preferred SLR scenarios defined for evaluating short-term, mid-term, and long-term impacts were selected as 1 foot, 2 feet, and 4 feet, respectively. These projected increases in sea level roughly correspond with NOAA's Intermediate scenario for the years 2030, 2050, and 2085 with a rather low exceedance probability (17%) and are within the range of the SLR scenarios recommended by Maine DOT for design of transportation infrastructure.

3.4 Site Vulnerabilities

The flood modelling data provided above in **Table 2 and Table 3** include scenarios for the Short Term, Mid Term, and Long Term sea-level rise scenarios. NOAA's Intermediate scenario mentioned above compared with these timeframes should be taken into consideration for the identified return periods as illustrated in **Table 4**.

Table 4: Flood Return Period

Event Return Period	**Percent Chance of Occurrence per Period**			
	5 Years	**10 Years**	**25 Years**	**50 Years**
100 Year Flood	4.9%	9.6%	22.2%	39.5%
500 Year Flood	1%	2%	4.9%	9.5%

The various site features have been summarized in **Table 5** for each facility, indicating the associated risk and flood scenario which result in inundation. Those elevations noted as 0 ft indicate an elevation similar to the identified feature of the facility. No elevations are noted in Table 5 where no inundation of the feature was identified (i.e., flood elevation is lower than that of the site feature). Below are the site-specific vulnerabilities based on our review of the property.

[2] https://tidesandcurrents.noaa.gov/publications/techrpt83_Global_and_Regional_SLR_Scenarios_for_the_US_final.pdf

Table 5: Site Elevations and Risks

Facility			*Inundation above Elevation of Facility*															
Description			*Present Day*				*Short Term Scenario*				*Mid Term Scenario*				*Long Term Scenario*			
		Elevation (ft) to NAVD88	*MHHW*	*HAT*	*1% Stillwater*	*BFE*	*MHHW*	*HAT*	*1% Stillwater*	*BFE*	*MHHW*	*HAT*	*1% Stillwater*	*BFE*	*MHHW*	*HAT*	*1% Stillwater*	*BFE*
			[ft]	*[ft]*	*[ft]*	*[ft]*	*[ft]*	*[ft]*	*[ft]*	*[ft]*	*[ft]*	*[ft]*	*[ft]*	*[ft]*	*[ft]*	*[ft]*	*[ft]*	*[ft]*
Pier	Lowest Horizontal	9.5 ft				1.5			0.5	2.5			1.5	4.5		1.6	3.5	7.5
	Lowest Deck or Adjacent Grade	10.5 ft				0.5				1.5			0.5	3.5		0.6	2.5	6.5
Wharf	Lowest Horizontal	9.5 ft				1.5			0.5	2.5			1.5	4.5		1.6	3.5	7.5
	Lowest Deck or Adjacent Grade	10.2 ft				0.8				1.8			0.8	3.8		0.9	2.8	6.8
Floating dock	Buoy Chain max elevation	9.5 ft				1.5			0.5	2.5			1.5	4.5		1.6	3.5	7.5
	Gangway footing	11.36 ft								0.64				2.64			1.64	5.64
Pavillion	Adjacent Grade	11 ft				0				1			0	3		0.1	2	6
	First Finished Floor	11.33 ft								0.67				2.67			1.67	5.67
	Lowest Opening	11.33 ft								0.67				2.67			1.67	5.67
Pump Station	Adjacent Grade	14.4 ft																2.6
	First Finished Floor	15.07 ft																1.93
	Lowest Opening	16.4 ft																0.6
Shoreline Protection	Top of riprap	11 ft				0				1			0	2		0.1	2	6
	Critial Elevation	16 ft																1
Harbor Master Office	Lowest Horizontal	9 ft				3			1	4			2	5		2.1	4	8
	First Finished Floor	9 ft				3			1	4			2	5		2.1	4	8
	Lowest Opening	9 ft				3			1	4			2	5		2.1	4	8
Yacht Club/ Restrooms	Lowest Horizontal	10.5 ft				1.5				2.5			0.5	3.5		0.6	2.5	6.5
	First Finished Floor	11 ft				1				2			0	3		0.1	2	6
	Lowest Opening	11 ft				1				2			0	3		0.1	2	6

Note: Facility elevations presented in this Table are referenced to NAVD88.

3.4.1 Pier and Wharf

Based on the present-day model for BFE of 11 to 15 feet which includes a wave height of 2 to 6 feet, the pier substructure and decking would be potentially impacted by high velocity wave action. In the case that any elements such as the abutment, stringers, etc., are not positively attached to subsequent load carrying members, dislodgement or delamination of material at the top of deck should be expected. Otherwise we would not expect any impact beyond minor delamination at the deck. A granite block or other material appears to be missing which would provide transfer of bearing loads from the pier (**See Photograph 10**). We were unable to view and assess other locations of the pier and wharf to note similar conditions at other locations due to limited access around the pier, which was provided only via the floating dock. The wharf can be expected to experience loss of smaller diameter crushed rock at the sub-base from washout resulting in deflection of the pavement and possibly complete delamination. Similar behaviour of the pier and wharf can be expected for future floor scenarios and the possibility of impact more likely as the return period for conditions representing the present day BFE decreases.

Site utilities which include water and electricity are exposed to wave action and inundation at the pier and the floating dock. A timber pipe bridge, which supports an electrical conduit, is located in the plane of the deck and near the floating dock entrance (**Photograph 18**). The structure does not appear to be securely fastened or designed to resist impact from wave action. In addition, the electrical cabinet and conduit at the wharf do not appear to be of waterproof construction (**Photograph 17**). Given the current position of the cabinet, we would expect some exposure to waver during the BFE for the Mid Term Scenario.

3.4.2 Floating Dock

The floating dock assembly consists of the gangway and the floats. The gangway attachment allows for rotation with a maximum limited by the elevation of the float at or beneath hinge elevation (**Photograph 14**). Normal operation does not appear to be influenced by the MHHW for all scenario. However, for the Short Term scenario, the gangway will be subjected to wave loading and uplift forces from the Stillwater elevation. In addition, the floats are moored to the perimeter piles at the south side of the pier via mooring chains. This attachment allows for a maximum mooring elevation roughly 15 inches below the top of pier deck (**Photograph 15**). Estimating some flexibility in this connection, the dock will be limited from traveling beyond elevation 9.5 feet (9 ft 6 inches) and begin to exert loading on the pier at water levels above this elevation. These potential vertical and lateral loadings already during the Present Day scenario under the BFE will continue to increase for subsequent scenarios based on the data in **Table 5**.

3.4.3 Site Structures

<u>Site 1</u>
The two structures observed at the site are the lobster pavilion and the sewer pump station. The estimated top of slab elevation at the pump station is elevation 14.4 feet. The BFE at this location as shown in Table 2 for the Present Day scenario is 15 feet; however, waves can be expected to dissipate at this inland distance to equal the 1% Stillwater elevation of 9 feet NAVD 88. The roughly 15 ft FFE of the building is also above the Short, Mid and Long Term scenarios' Stillwater elevation. At Long Term scenarios, above-ground architectural and structural elements will likely be impacted, during which coatings, coverings and their fastenings will be subjected to moisture. Also, openings such as doors, windows, and louvers will be a means for moisture intrusion. Depending on the interior architectural finishes, some delamination and material degradation can be expected. Given the elevation of the pump station and wet well hatch, inundation at the long term and possibly the Mid Term scenario is of concern. During our site visit we were unable to confirm whether these elements are sealed systems due to lack of access.

Similar concerns exist for the lobster pavilion, which sits at a FFE of approximately 11.3 feet NAVD88. During the Long Term Scenario, flooding of the area can be expected from the design Stillwater elevation. No serious wave action however is expected at this inland location for this scenario. We expect more frequent inundation of the area during the Long Term scenario for the HAT. For the type of construction, minimal material degradation is expected from inundation, however factors such as the existence of horizontal ties, reinforced and/or grouted cells, or type of footing will define the behaviour of the structure from moisture intrusion, which can be expected for the Long Term scenario.

<u>Site 2</u>
During our preliminary walkthrough of the park/open space, no notable deficiencies related to the buildings or the site features were documented. Site structures, such as the Harbor Master's office and the yacht club building were visually evaluated from the exterior only, revealing no obvious defects which would compromise the structural integrity of the buildings. The FFE of the Harbor Master's office (**Photograph 28**) sits at roughly elevation 9 feet. Given the structure's proximity to the coast, we expect wave action to be a contributing factor in the hydrodynamic loading. The 1% Stillwater elevation of 9 feet and 6 additional feet at the BFE indicates this structure already lies within the 100-year return period for the Present Day scenario. Subsequent scenarios exhibit values which increase for all data sets with the Long Term BFE exceeding the structure's roof height. It is noteworthy that the HAT of elevation 9.1 feet roughly coincides with the current site elevation for the Mid Term scenario (See Table 3).

The building which houses the yacht club and public restrooms is constructed with portions of the structure's FFE near grade and others with floor elevations 1.5 to 2 feet above the grade (**Photograph 29**). For the Present Day scenario, portions of the building, in particular the office and restrooms, are above the 1% Stillwater elevation but lie approximately at the BFE of 11 feet.

For the Present Day scenario, the foundation of both structures will be inundated and the building envelope will be subjected to wave action primarily dependant on its inland location. Information was not readily available regarding the type of foundation, specifications of the building design or framing system. Similar to Site 1, the possibility of damage to structural components and architectural finishes, at a minimum, can be expected for all scenarios with increasing severity advancing into the future. For the Long Term scenario, a combination of SLR and 100-year storm events creates a risk for both the Harbor Master Office and the Yacht Club to high velocity wave action.

3.4.4 Shoreline Protection

Site 1

Shoreline protection is provided by a revetment ranging in elevation from about 11 feet to 12 feet. Large diameter (roughly 1.5 to 4 ft) riprap is provided along the perimeter of the site extending from below the low tide level to the top of grade (**Photographs 24 & 25**). The estimated slope is a maximum of 3 to 1, horizontal to vertical, and gradation appears to be suitable based on condition of slope. No signs of material degradation or slope instability or piping were noted. Based on existing conditions, the risk of overtopping during the Present Day scenario is relatively low. Overtopping is more likely for the Short and Mid Term scenarios. Some landward flooding in the range of 2 to 7 feet will occur during the overtopping during these scenarios but it will not undermine the revetment. Under wave attack, randomly placed riprap will experience some settlement and readjustment; however, the risk of wide-scale riprap slope failure appears low. The risk of localized scour or dislodging of riprap is low and given their inherent stability they will likely require minimal remediation for the Short Term and future scenario.

Site 2

Shoreline protection at this location was not directly assessed however appears to be a stacked granite sea wall which may also support the wharf. Delamination of material was not observed, and structure appears to function as intended.

4.0 RECOMMENDATIONS

4.1 General Recommendations

In accordance with American Society of Civil Engineers / Structural Engineering Institute Standard 24 – Flood Resistant Design and Construction (ASCE 24), existing structures that sustain substantial damage, or that are substantially improved, are treated as new construction. This standard considers damage beyond routine maintenance or otherwise minimal damage following an event, which nonetheless requires major improvements and even applies to structures classified as pre-FIRM. For new construction **we recommend, in light of the forecasted increase in water levels and the schedule for these events in relationship to the life of the structure, design should be based on the either BFE plus 2 feet of freeboard, the DFE, or 500-year event, whichever is higher**. It is understood that local requirements coupled with available resources will dictate the ability for the communities to incorporate proactive designs. The following recommendations are provided with regard to areas of the site which fall within a special flood hazard area:

- All new construction, substantially improved, and substantially damaged buildings must be elevated on pilings, posts, piers, or columns so that the bottom of the lowest horizontal structural member of the lowest floor is at or above the design BFE plus 1 to 2 feet of freeboard, per American Society of Civil Engineers / Structural Engineering Institute Standard 24, Flood Resistant Design and Construction (ASCE 24).
- For inland building/structures, the First Floor Elevation should be above the 1% Stillwater elevation plus 1 to 2 feet of freeboard.
- The foundation system must be anchored to resist flotation, collapse, lateral movement due to wind and water loads acting simultaneously on all components of the building.
- Erosion control structures shall not be attached to the building or its foundation. Riprap or revetment should be installed at a minimum height above the 1% Stillwater elevation but below the BFE, where some overtopping is allowed.
- Use of flood damage-resistant materials above the BFE per ASCE 24 and the local Building Code.
- Slab on grade construction in this zone is not permitted and should be avoided.

- Electrical, heating, ventilation, Plumbing and Air Conditioning Equipment should be located on the **landward side of any building and/or behind structural elements. They must be elevated and designed to prevent flood waters from entering and accumulating in components during flooding**.
- Install shutoff and isolation valves on water and sewer lines that extend into the flood-prone areas.

This list is not comprehensive but rather apply to site features observed during our site visit. There may exist other relevant items addressed in any of the above-mentioned design standards which are applicable for the site at a future date. We recommend a detailed site assessment be performed during the design stage to ensure implementation of all applicable items.

4.2 Site Specific Recommendations

Although the risks, vulnerabilities, and associated recommendations addressed herein are in reference to features located within the property limits of the Town Dock, there may be features of similar construction in close proximity and exposed to similar risks as described in this report but fall outside the limits of assessment. **We recommend that these sites and features undergo a similar assessment with the assumption that similar or greater risks may apply.** The following are recommendations for the features identified at risk for Middle Pier and associated project specific areas.

4.2.1 Pier and Wharf

The following recommendations are provided in reference for the **Present Day and all future scenarios** for flood values provided in **Table 2 and 3** above:

- A detailed structural assessment is recommended for the deck and substructures. Confirm positive attachment of all structural members to their substrate or load-bearing elements. Straps should be designed and incorporated for purposes of hold-down against wind and water loads.
- Utilities should be properly secured to resist design wind and water loading or relocated above the design flood elevation as specified in ASCE 24. Watertight and stainless-steel electrical fixtures should be incorporated. Confirm that all building utilities are placed 2 feet above the flood elevation and/or sealed from inflow of flood water.

The following additional recommendations are provided in reference for the **Short Term scenario** for flood values provided in **Table 2 and 3** above:

- Subgrade conditions under paved areas of the wharf should be verified, voids should be filled, and the subgrade should be compacted per local requirements.

The following recommendations are provided in reference for the **Mid Term scenario** for flood values provided in **Table 2 and 3** above:

- The structure should be re-evaluated based on the current design loading with regard to wind and wave action. Subgrade conditions under paved areas of the wharf should be verified, voids should be filled, and the subgrade should be compacted per local requirements. Damaged or inadequate elements of the structure should be replaced or repaired as needed.

The following recommendations are provided in reference for the **Long Term scenario** for flood values provided in **Table 2 and 3** above:

- Based on the elevated wave action, we recommend installation of a reinforced concrete deck and piles which is able to support design loading independent of the supportive conditions below the granite blocks and at an elevation to accommodate requirements of ASCE 24, Flood Design Manual

For future flood scenarios, it may be necessary to reconstruct the wharf and/or pier to accommodate sea level rise and increased risk of damage due to more frequent events. Raising the elevation of the breakwater, which provides protection for the coastline, is also a consideration. Modelling of the scenario which incorporates an elevated breakwater may be valuable in providing comparative values for a repair/replacement feasibility study. This analysis is not a part of this assessment.

4.2.2 Floating Dock

The following recommendation is provided in reference to the **Present Day scenario** with regard to construction of the floating dock assembly:

- Conduct a detailed evaluation for the mooring system of the floating dock and the buoyancy and wave loads imposed on the float and wharf structures due to future water level rise and storms. Design and install separate mooring piles for the floats to avoid attachment to the pier. Piles should be installed to accommodate a BFE of at least the Mid Term condition with the addition of 2 ft of freeboard.

The following recommendation is provided in reference to the **Short Term and all future scenarios** with regard to construction of the floating dock assembly:

- Confirm the gangway attachments are sufficient to resist the design loading and repair or replace as needed.

4.2.3 Site Structures

The basis of our recommendation for buildings or other structures is the inland location of the structure and behaviour of the event at that location. For structures located in close proximity to the shoreline, wave impact from the BFE is a threat and the possibility of immediate damage more likely. For inland structures where wave action has subsided, concerns of static flooding are more prevalent. For these cases, we are concerned about inundation, such as for the Stillwater and MHHW at the FFE, where the usefulness of the structure is compromised.

The following recommendations are provided in reference to the **Present Day scenario** provided in Table 2 and 3 above:

- Based on the BFE and associated wave action, the Harbor Master's office should be evaluated to confirm ability to resist the design loading from wave and wind and retrofitted based on results of this analysis.

The following recommendations are provided in reference to the **Short Term scenario** provided in Table 2 and 3 above:

- Based on the design Stillwater and its influence on the Harbor Master's office, re-evaluation of the structure will be required based on its functionality (Serviceability requirements). The assessment will be per current design standards to estimate adequacy to support the intended design loads. Following analysis, a likely recommendation is to retrofit or replace the existing building. This may constitute the minimum, whereas relocation may be recommended based on cost.

The following recommendations are provided in reference to the **Mid Term scenario** provided in Table 2 above:

- Install erosion control measures at the perimeter of the Lobster Pavilion against scour.

The following recommendations are provided in reference to the **Long Term scenario** provided in Table 2 above:

- Where the Sewer Pump Station and associated wet well are impacted by future scenarios, incorporation of a flood-proof design is recommended such as stem walls extending above the design flood elevation as dry floodproofing, sealed openings, and sealed or gasketed floor/ground openings. Confirm that all building utilities are placed above the design flood elevation and/or sealed from inflow of flood water. In general, existing structures and their foundations should be assessed for the applicable design loading. Repairs, retrofits, or improvements should be per the local

Building Code and performed under recommendation and guidance of a Professional Engineer Registered in the State of Maine.

- The Yacht Club and associated facilities should be evaluated for flood and wind loading per the current design standard. The pressure to relocate the structure due to frequent inundation of the MHHW is not predicted, but nonetheless flooding in expected for this period during the design Stillwater with some influence of residual wave action based on the structure's proximity to the shore. Some repair effort is foreseen of minimal to substantial improvements for the scenario, and considering the BFE and Stillwater elevations, it will be practical to re-build the structure completely either with a FFE above the BFE and 2 feet of freeboard. Design for improvements should be per the local Building Code and performed under recommendation and guidance of a Professional Engineer Registered in the State of Maine.

4.2.4 Shoreline Protection

Based on our field observations and analysis data for the Present Day and Short Term scenario, we are of the opinion that minimal damage should be expected or otherwise no catastrophic failure of the revetment. Preliminary calculations, given certain assumptions, indicate the current riprap size can be expected to perform as expected for these events. For subsequent higher scenarios, we would expect increased overtopping which would allow for upstream flooding. The behaviour of the revetment during extreme events is dependant heavily on the average rock diameter, the height of revetment in relation to the wave height, and the layer thickness. Behaviour of the structure under extreme conditions (excessive wave height) can only be estimated with confidence by means of hydraulic model tests. We therefore recommend the following:

- Verification of the layer thickness is at least 2.5 feet.
- Sample verification of the D_{50} size of material to be 1.3 ft or greater.
- For extreme events or wave height is beyond 5 feet, a hydraulic model test is recommended for existing conditions.

5.0 OPINION OF PROBABLE CONSTRUCTION COSTS

The costing information provided below corresponds with our recommendations for remedial action for the corresponding events as outlined in **Table 2 and 3** of this report. These estimated costs include the associated design and engineering services where applicable. In **Table 6** is a summary of the estimated cost for repair or replacement of the identified vulnerabilities. A cost savings may also be expected for combined effort for items similar in nature, for example, replacing the electrical cabinet while updating and/or securing electrical conduits. We have not considered this variable in our values. Where a complete replacement option is provided, this option and associated costs may be implemented sooner depending on the priorities and funding available to the City. Costing for the referenced scenario represents summation of all non-complementary improvements. That is, where other repairs, such as intermediate retrofitting, are performed during preceding scenarios the associated costs become additive. All costs are based on present value without inflation. Provided below is a more detailed description of the items included for the associated risk scenario.

5.1 Present Day Scenario

Pier and Wharf:

The following costs should be expected to accommodate events associated with the Present Day scenario:

- Replace current electrical cabinet with stainless steel watertight cabinet. Design and Construction **$12,000.**
- Utilities should be properly secured to resist design wind and water loading or relocated above the design flood elevation as specified in ASCE 24. Design and Construction **$175.000.**

Table 6: Repair / Replacement / Retrofitting Costs

Facility	***Present Day***	***Short Term***	***Mid Term***	***Long Term***
Pier / Wharf	$297,000	$332,000	$812,000	$2,687,000
Floating Dock	$70,000	$175,000	$175,000	$175,000
Pavilion			$150,000	$200,000
Pump Station				$450,000
Shoreline Protection				$250,000
Harbor Master Office	$250,000	$625,000	$625,000	$625,000
Yacht Club/ Restrooms				$450,000
TOTAL:	**$617,000**	**$1,132,000**	**$1,762,000**	**$4,837,000**

- Confirm positive attachment of all structural members to their substrate or load-bearing elements. Straps and hold-downs should be designed and incorporated for the purpose to resist wind and water loads. Design and Construction **$110,000.**

Floating Dock:

- Moor the float to independent float piles or using mooring chains/ropes anchored to the seabed. Design and Construction **$70,000.**

Site Structures:

<u>Harbor Master Office</u>

- Evaluate structure per current design standards to support the intended design loads. Retrofit as needed. Design and Construction **$250,000.**

5.2 Short Term Scenario

Pier and Wharf:

The following costs should be expected to accommodate events associated with the Short Term scenario:

- Replace current electrical cabinet with stainless steel watertight cabinet. Design and Construction **$12,000.**
- Utilities should be properly secured to resist design wind and water loading or relocated above the design flood elevation as specified in ASCE 24. Design and Construction **$175.000.**
- Confirm positive attachment of all structural members to their substrate or load-bearing elements. Verify subgrade conditions, fill and/or replace material and compact, as needed. Design and Construction **$145,000.**

Floating Dock:

- Moor the float to independent float piles or using mooring chains/ropes anchored to the seabed. Design and Construction **$70,000.**
- Confirm the gangway attachments ability to resist the design loading and repair or replace as needed. Design and Construction **$105,000**

Site Structures:

Harbor Master Office

- Re-evaluation of the structure per current design standards to support the intended design loads. Retrofit, replace or relocate the existing structure. Design and Construction **$625,000.**

5.3 Mid Term Scenario

This section exhibits costs which are expected due to the need for substantial improvements, however some of these actions are recommended as early as the Present Day scenario.

Pier and Wharf:

- Re-evaluate the structure per current design standards to estimate adequacy to support the intended design loads. Following analysis, a likely recommendation is to reinforce or replace damaged, deteriorated or missing elements of the wooden pier to include railing, posts, decking and piles and in addition, provide positive attachment of all elements per the International Building Code with State of Maine amendments. This may constitute the minimum, whereas complete replacement may be recommended based on performance results. Design and Construction **$625,000.**
- Replace current electrical cabinet with stainless steel watertight cabinet. Design and Construction **$12,000.**
- Utilities should be properly secured to resist design wind and water loading or relocated above the design flood elevation as specified in ASCE 24. Design and Construction **$175.000.**

Floating Dock:

- Moor the float to independent float piles or using mooring chains/ropes anchored to the seabed. Design and Construction **$70,000.**
- Confirm the gangway attachments ability to resist the design loading and repair or replace as needed. Design and Construction **$105,000**

Site Structures:

Harbor Master Office

- Re-evaluation of the structure per current design standards to support the intended design loads. Retrofit, replace or relocate the existing structure. Design and Construction **$625,000.**

Lobster Pavilion

- Install erosion control measures. Design and Construction **$150,000.**

5.4 Long Term Scenario

This section exhibits costs which are expected due to the need for substantial improvements, however some of these actions are recommended as early as the Present Day scenario and could lead to decreased costs later.

Pier and Wharf:

- Install a reinforced concrete deck which capable of supporting design loading independent of the subgrade conditions above the granite blocks and at an elevation to accommodate requirements of ASCE 24, Flood Design Manual. Design and Construction **$2,500,000.**

- Replace current electrical cabinet with stainless steel watertight cabinet. Design and Construction **$12,000.**

- Utilities should be properly secured to resist design wind and water loading or relocated above the flood elevation as specified in ASCE 24. Design and Construction **$175.000.**

Floating Dock:

- Moor the float to independent float piles or using mooring chains/ropes anchored to the seabed. Design and Construction **$70,000.**

- Confirm the gangway attachments ability to resist the design loading and repair or replace as needed. Design and Construction **$105,000**

Site Structures:

Pump Station

- Verify and incorporate flood-proof design and construction for all utilities and opening at site. Provide repairs, retrofits, or improvements as needed. Design and Construction **$450,000.**

Lobster Pavilion

- Abandon existing Lobster Pavilion in flood zone and build a new lobster pavilion on higher elevation 2 ft above the design flood elevation. Design and Construction **$200,000**.

Harbor Master Office

- Re-evaluation of the structure per current design standards to support the intended design loads. Retrofit, replace or relocate the existing structure. Design and Construction **$625,000.**

Yacht Club

- Re-build the structure completely at higher elevation with sustainable design per local and national design standards. Design and Construction **$450,000.**

Shoreline Protection:

- Raising Riprap to an appropriated elevation (approximate 4 – 5 ft) based on the design water levels and increased wave heights. Design and construction: **$250,000.**

6.0 QUALIFICATIONS OF THE REPORT

The DMR should understand that our observations may be inconclusive, or it may not be possible to identify a definitive cause of distress based on a structural inspection and visual observations alone/without further testing. The recommendations are made based on these limitations.

The "Opinion of Probable Construction Costs" is made on the basis of Wood PLC's judgment, as experienced and qualified professionals generally familiar with the construction industry. However, since Wood, PLC has no control over the cost of labor, materials, equipment, or services furnished by others, or over the construction contractor's methods of determining prices, or over competitive bidding or market conditions, Wood cannot, and does not, guarantee that proposals, bids, or actual construction cost will not vary from the Opinion of Probable Construction Costs prepared by Wood PLC. We have attempted to consider all aspects of the work and site conditions, based on information made available to us at this stage of the project. Costs will be modified during subsequent stages of project execution, as the level of project definition increases. All costs are based on actual costs as provided by RS Means Costworks 2018, additional or other specified suppliers vendors and contractors.

7.0 CLOSING

Wood appreciate the opportunity to provide these services to DMR on this project. Please contact us with any questions or comments.

Sincerely,
Wood Environment & Infrastructure Solutions, Inc.

Tirrell Day, PE
Senior Structural Engineer

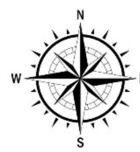
D. Todd Coffin
Associate Project Manager

Attachments: Appendix A - Photolog
Appendix B – Inundation Maps and Cross Sections

Appendix A - Photolog for Middle Pier, Rockland ME

By: *T. Day* ***Date:*** *03SEPT2019* ***Reviewed:*** *K. Sun* ***Date:*** *05DEC2019*

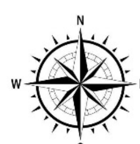
Photograph No. 1:



Comment:

Overview of Site 1.

Photograph No. 2:



Comment:

Overview of Site 2.

By: T. Day Date: 03SEPT2019 Reviewed: K. Sun Date: 05DEC2019

Photograph	Comment
Photograph No. 3: 	**Comment:** **View of parking area near park.**
Photograph No. 4: 	**Comment:** **View of parking area near park.**

By: *T. Day* *Date:* *03SEPT2019* *Reviewed:* *K. Sun* *Date:* *05DEC2019*

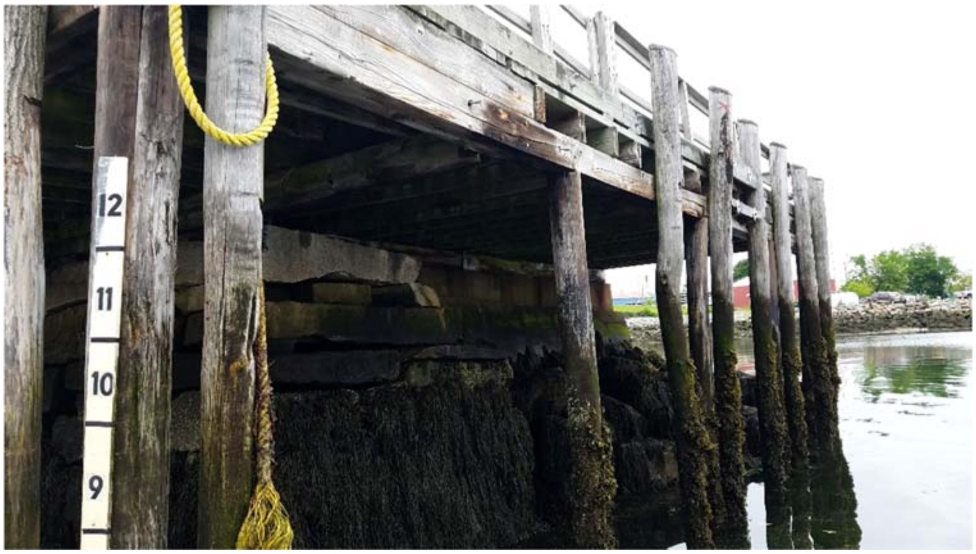
Photograph No. 5:	Comment:
	View of wharf from above.
Photograph No. 6:	**Comment:**
	View of South Pier from above.

By: *T. Day* *Date:* *03SEPT2019* *Reviewed:* *K. Sun* *Date:* *05DEC2019*

Photograph	Comment
Photograph No. 7: 	**Comment:** **View of West Pier from above**
Photograph No. 8: 	**Comment:** **Close-up of timber members at pier.**

By: *T. Day* ***Date:*** *03SEPT2019* ***Reviewed:*** *K. Sun* ***Date:*** *05DEC2019*

Photograph No. 9:



Comment:

View facing northeast at front of pier and wharf.

Photograph No. 10:

Comment:

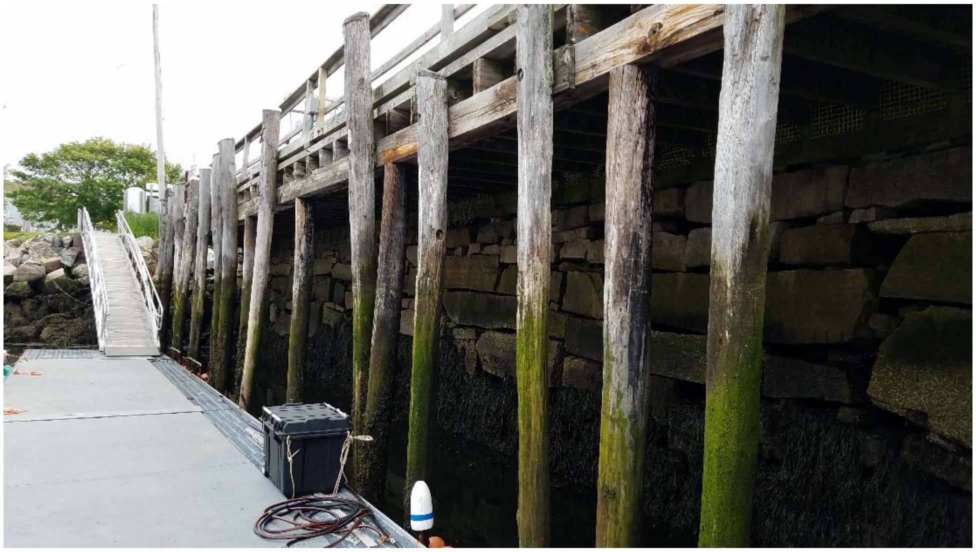
View facing north at south side of wharf and pier.

View of typical construction.

1. **Possible bearing element missing for transfer of loading to subsequent support member.**

By: T. Day *Date:* 03SEPT2019 *Reviewed:* K. Sun *Date:* 05DEC2019

Photograph No. 11:



Comment:

View of south pier at wharf and floating dock.

Photograph No. 12:

Comment:

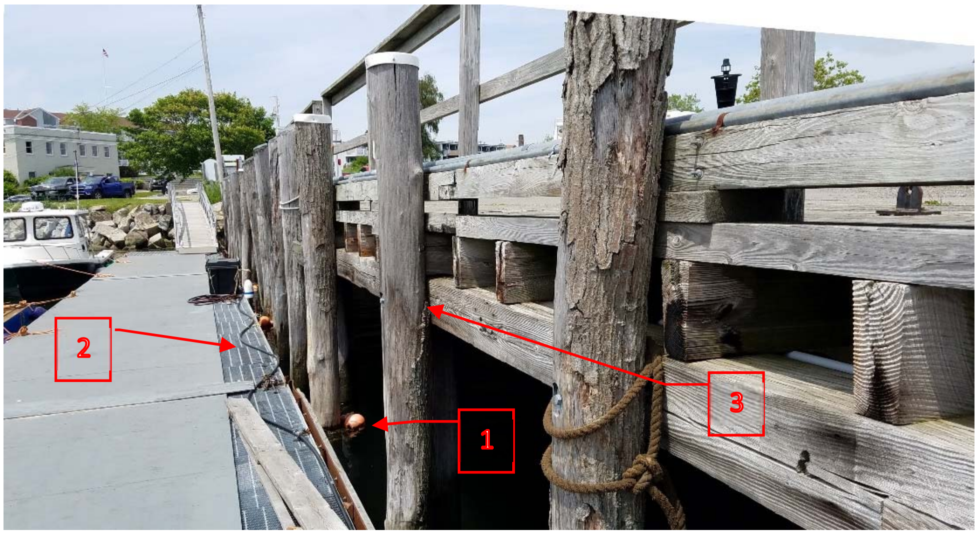
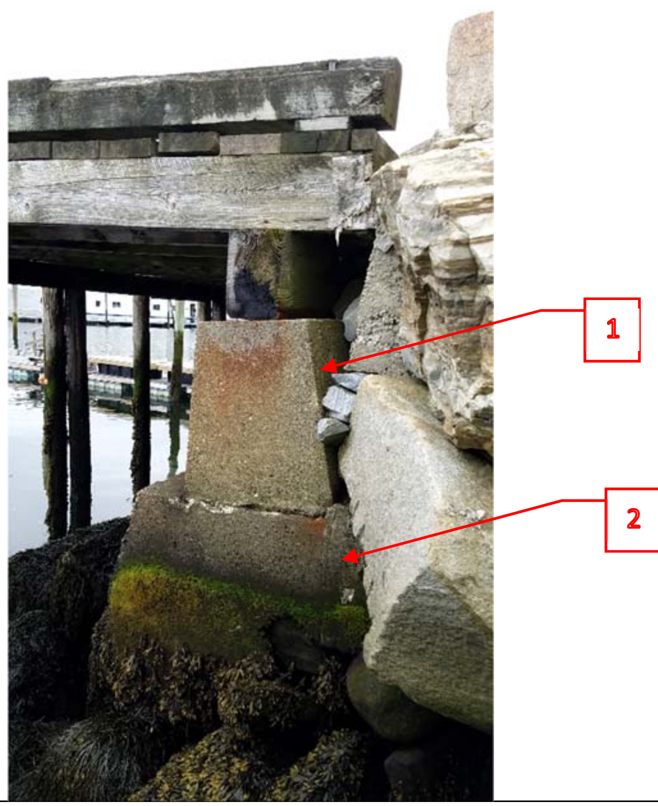
View of floating dock gang way attachment to concrete footing.

By: T. Day Date: 03SEPT2019 Reviewed: K. Sun Date: 05DEC2019

Photograph No. 13:	Comment:
	View from above at location in Photograph 12. Attachment of sill and cover plate.

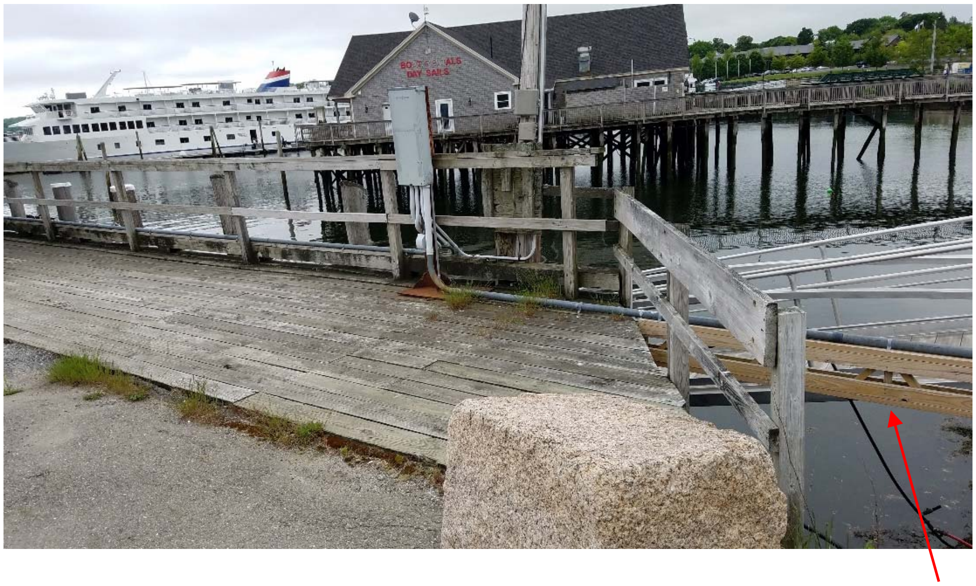
Photograph No. 14:	Comment:
	Side view of gang way attachment facing north.

By: *T. Day* ***Date:*** *03SEPT2019* ***Reviewed:*** *K. Sun* ***Date:*** *05DEC2019*

Photograph	Comment
Photograph No. 15: 	**Comment:** **1. Mooring Chain attachment at float.** **2. Floating deck water supply line.** **3. Limits of floating dock mooring chain at high water level.**
Photograph No. 16: 	**Comment:** **View of abutment support of pier at north side.** **1. Stem support** **2. Footing**

By: *T. Day* *Date:* *03SEPT2019* *Reviewed:* *K. Sun* *Date:* *05DEC2019*

Photograph No. 17:



Comment:

View of pier and pipe bridge for electrical conduit which is vulnerable to wave action.

Photograph No. 18:

Comment:

Close-up of electrical conduit and pipe bridge attachment to pier as noted in previous photo.

By: T. Day Date: 03SEPT2019 Reviewed: K. Sun Date: 05DEC2019

Photograph No. 19:	Comment:
	View of sewer pump station and wet well cover.

Photograph No. 20:	Comment:
	Rear view of sewer pump station building.

By: *T. Day* *Date:* *03SEPT2019* *Reviewed:* *K. Sun* *Date:* *05DEC2019*

Photograph No. 21:

Comment:

View of lobster cooking station near entrance of property.

Photograph No. 22:

Comments:

View of roof structure; open gable.

By: *T. Day* ***Date:*** *03SEPT2019* ***Reviewed:*** *K. Sun* ***Date:*** *05DEC2019*

Photograph No. 23:	**Comments:**
	**View at open side of lobster cooking station.**
Photograph No. 24:	**Comments:**
	**View of revetment at south side of property.**

By: *T. Day* *Date:* *03SEPT2019* *Reviewed:* *K. Sun* *Date:* *05DEC2019*

Photograph No. 25:	Comments:
	View of riprap revetment at north side of property.
Photograph No. 26:	**Comments:**
	View of sea wall at harbor park near Harbormaster's office.

By: *T. Day* ***Date:*** *03SEPT2019* ***Reviewed:*** *K. Sun* ***Date:*** *05DEC2019*

Photograph No. 27:	Comments:
	View facing north at series of docks for large carriers.

Photograph No. 28:	Comments:
	View of Harbormaster office.

By: *T. Day* *Date:* *03SEPT2019* *Reviewed:* *K. Sun* *Date:* *05DEC2019*

Photograph No. 29:

Comments:

View of building housing the yacht club and public restrooms.

Photograph No. 30:

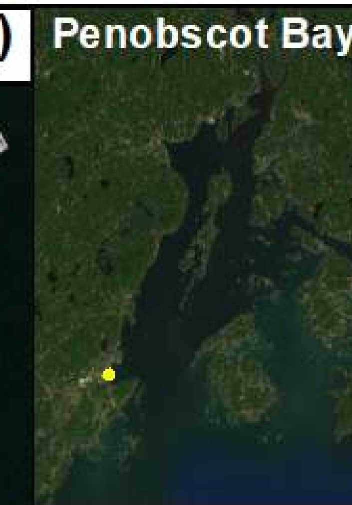
Comments:

View of paved area above wharf. Close-up of deflection cracking.

By: *T. Day* ***Date:*** *03SEPT2019* ***Reviewed:*** *K. Sun* ***Date:*** *05DEC2019*

Photograph No. 31:	Comments:
	View of cracks and surface delamination at pavement for wharf.
Photograph No. 32:	**Comments:**
	Additional signs of failing at paved area. Signs of previous repair work.

Appendix B – Inundation Maps



1. Maps indicate the 1%-annual-chance flood zones and base flood elevations.

2. This does not constitute a revision to the FEMA FIRM map which is done through FEMA's Letter of Map Revision process for which additional analysis and/or modeling may be required.

3. Flood mapping was developed for planning purposes only. No other use of this map should be made.

4. Elevations in reference to vertical datum NAVD88

Rockland Transects
AE
VE
X
N

Zone AE : Coastal flood zone. Base Flood Elevations determined.

Zone VE : Coastal flood zone with velocity hazard (wave action). Base Flood Elevations determined.

Zone X : Areas determined to be outside the 1% annual chance floodplain.

0 50 100 200 Feet

WOODS HOLE GROUP
A CLS COMPANY