INNOVATIVE DESIGN OF INTEGRAL PIERS PROVE COST-EFFECTIVE FOR THE MARTIN'S POINT BRIDGE

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ABSTRACT

Stretching from Falmouth to Portland, Maine, Martin's Point Bridge serves an important role in connecting these two communities. A Design-Build effort focused on cost-efficiency and multimodal safety has transformed the structure into the first multi-integral pier bridge in the state to use large precast/prestressed concrete girders. This accomplishment required supporting the 1,286-foot-long concrete girder bridge with eight integral piers and one traditional pier, located at the only deck expansion joint. Iterative designs and sensitivity analyses of materials and foundation conditions allowed the team to confidently construct these piers before the superstructure design was finalized. This paper explores the design considerations for the AASHTO Strength Limit States, the differing site conditions, varying pier heights, specific project requirements, and the constructability challenges anticipated and encountered during construction. Additionally, the paper discusses the challenges associated with designing for seismic and ice loads, the flexibility behavior and interactions at the integral connections, and the variation in material properties and stiffness. The innovative design of multiple integral piers with precast/prestressed girders at the Martin's Point Bridge proved to be an economical structure for the Maine Department of Transportation and may serve as a model for future projects.

Keywords: Construction, Creative/Innovative Solutions and Structures, Design-Build

INTRODUCTION

Stretching from Falmouth to Portland, Maine, Martin's Point Bridge serves an important role in connecting these two communities. A Design-Build effort focused on cost-efficiency and multimodal safety has transformed the structure into the first multi-integral pier bridge in the state to use large precast/prestressed concrete girders. Iterative designs and sensitivity analyses of materials and foundation conditions provided a 1,286-foot-long concrete girder bridge with eight integral piers and one traditional pier, located at the only deck expansion joint.

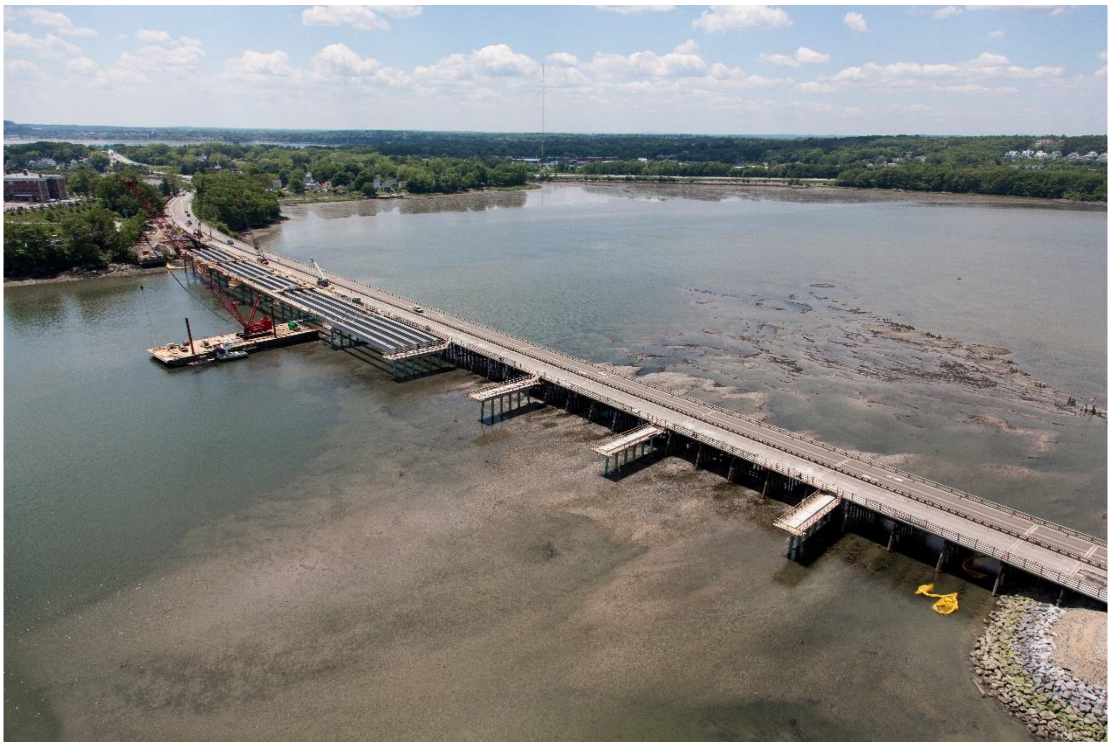

Figure 1: Aerial view of the Martin's Point Bridge under construction

DESIGN-BUILD

The Maine Department of Transportation (MaineDOT) issued the replacement of the Martin's Point Bridge as a Design-Build project. CPM Constructors of Freeport, Maine was awarded the project as the Design-Builder and VHB as the design engineer. One of the key advantages to the CPM-VHB proposal was the use of integral piers. Extensive preliminary engineering went into analyzing this structure to ensure that it could be accomplished. The integral piers significantly reduced substructure costs by allowing fewer and smaller diameter pipe piles. The estimated construction cost is $21.3 million, which includes 1500 feet of approach work and removal of the existing 1400 foot-long bridge. Construction began in October 2012 and is scheduled for completion at the end of December 2014. The bridge was opened to traffic in June of 2014.

The Request for Proposal (RFP) issued by MaineDOT outlined several major requirements for the project. These included geometric constraints, wetland impact limitations, maintenance of traffic during construction, and other explicit bridge details. In particular, all superstructure elements with flanges greater than 2.5” wide must be angled at 45 degrees to prevent bird roosting with the exception of New England Bulb Tees (NEBT). Clearly, a concrete superstructure was required.

DESIGN

SPAN ALTERNATIVES

To provide a clean and aesthetic appearance, a minimum slenderness ratio of 17 was desired. The slenderness ratio compares the length of the span between centerline of piers and/or abutments to the total superstructure depth. The superstructure depth includes blocking, exterior girder depth, and depth of the deck or sidewalk at the fascia. Three span alternatives were considered during the preliminary design process to avoid conflicts with the existing bridge substructure and to maintain the hydraulic opening. Northeast Extreme Tee Beam (NEXT) beams were considered for spans of 80 feet. NEBT beams were considered for spans of 130 feet. A 78” deep post-tensioned concrete beam option was considered for spans over 150 feet. The shape of the post-tensioned beam was based on the NEBT, with a wider web to accommodate post-tensioning strands.

The standard NEBT option with 130 foot spans provided the best balance between superstructure depth, slenderness ratio, wetland impacts, and hydraulic clearance. A ten (10) span configuration was selected, which reduced the number of piers from 44 in the existing condition to 9 in the proposed condition. The span layout of the new bridge provides the required horizontal hydraulic opening, maintains the existing navigation channel, and avoids the existing substructures. In addition, the longer 130’ spans allowed for fewer spans, fewer piers, and therefore fewer piles. Minimizing piles saved both time and money.

BEAM DESIGN

Iterative analyses provided an optimized design consisting of five beam lines 71 inches deep. The fifty beams were grouped and consolidated so only three different strand patterns would be needed. (See Table 1). This allowed for two beams to be fabricated in a casting bed at one time.

Pattern	# Beams	# Strands	# Draped Strands
1	15	52	10
2	30	48	8
3	5	44	8

Table 1: Strand patterns

The alignment and roadway width requirements created many challenges for the beam layout. An offline alignment was developed to maintain traffic on the existing bridge during construction. Horizontal curves at both ends of the bridge match into the existing alignment on the approaches. Additionally, a taper in Spans 1 and 2 was used to accommodate a turn lane and resulted in a varying bridge width (58’-4” maximum, 53’-3” minimum). The horizontal curves resulted in varying overhangs along most of the bridge and the taper necessitated splayed beams in the first two spans. Deck overhangs at exterior girders varied from 3’-6” to 4’-5” maximum. The combination of the horizontal curve and taper in Spans 1 and 2 created varying beams lengths, so Piers 1 and 2 were skewed to create more consistent lengths within the two spans. Figure 2 shows the typical superstructure section.

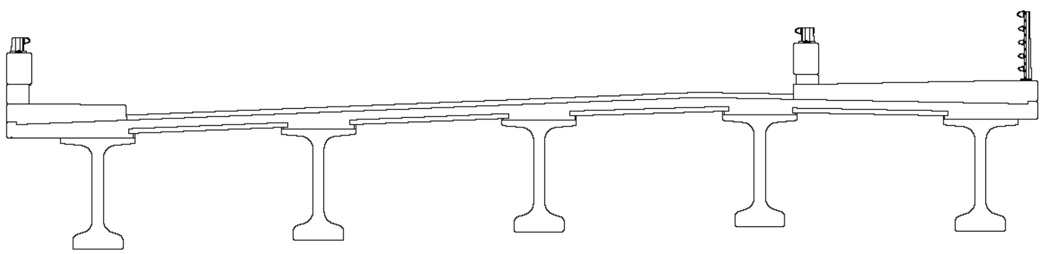

Figure 2: Superstructure Section

Draped or harped strands and a 10 ksi concrete strength were needed to accommodate the varying beam spacing (from 12’-4” maximum to 11’-4” minimum) and to meet design requirements. MaineDOT limits the designer to the severe exposure tensile limit for Service III limit state at the bottom of the beam and requires a 25% increase to the truck load for the Strength I limit state. Additionally, the beams are integrally connected to the substructure, so they also need to accommodate additional stresses from induced thermal moments at the pier connections. These moments create a stress at the bottom of the beam that combine with stresses from the beam design. Therefore, preliminary beam designs needed excess prestressing to provide zero tension the bottom of the beams once the integral pier stresses were verified in final design.

A positive moment connection is required at the piers to design the beams as continuous for live load. Since all strand locations are utilized for the strength design, there is not enough space to embed reinforcing into the bottom of the beam ends. Instead, the prestressing strands extend into the diaphragm to make the positive moment connection. Alternate projection patterns are used at beam ends to fit the strand extensions without interference within the same diaphragm (See Figure 3). Prestressing strands are 0.6-inch diameter, with an ultimate strength of 270ksi.


Figure 3: Strand Projections at Beam Ends.

The integral pier configuration locks down the superstructure to the substructure sufficiently bracing the system so that intermediate diaphragms are not required. Temporary intermediate cross frames were used only during construction before the integral pier diaphragms were in place. (See Figure 4). AASHTO live load distribution factors were used for the beam design.


Figure 4: Temporary Intermediate Pipe Cross Frames.

The use of precast deck panels, a range of anticipated beam cambers, and the superelevation transitions on the bridge required variable projection heights for the horizontal shear reinforcing steel to ensure minimum penetration into the deck and clear cover requirements. The precast deck panels are 3½" thick, with a design concrete strength of 6.5ksi. The panels are made composite with a 5" cast-in-place concrete topping

BEAM FABRICATION CHALLENGES

The designers worked with a local fabricator to ensure the combination of a higher strength concrete mix and draped strands was feasible during fabrication. This partnership created a better understanding of the marriage between design requirements and fabrication capabilities. It also allowed greater flexibility over the design elements and prompt responses to fabrication and construction issues.

Beams vary in length, but are generally about 125 feet long and upwards of 120,000 pounds (150 pcf). Early discussions with the fabricator and Contractor as well as careful consideration by all was required to ensure the beams could be fabricated, transported, and handled appropriately onsite with equipment available.
The fabricator was familiar with MaineDOT's concrete mix specifications, but had limited experience with mix designs requiring a compressive strength of 10ksi. Release strengths varied from 6.5 ksi to 7.5 ksi, depending on span and beam line. In anticipation of possible fabrication defects, the fabricator provided a report detailing repair procedures to amend some standard fabrication issues. The report detailed repairs for end cracks in webs and flanges, corner spalls, and non-confined bearing area spalls. The repair document saved time as minor repairs could be completed immediately.

To expedite fabrication, two beams were poured simultaneously in the casting bed when strand patterns were identical. This procedure posed an issue with the draped strands, though, since the strands were pulled over a longer distance and subject to more friction losses compared to one beam at a time. To achieve minimum elongations, the strands needed to be over-pulled. Over-pulling the strands required a variance from several specification limitations, including reducing initial pull forces at the dead end, reducing the algebraic difference between gauge pressure and elongation, and allowing additional force above the calculated gauge pressure. Independent calculations confirmed that allowing these variations did not adversely affect the beams.

Anticipated cambers were provided on the plans. To verify these theoretical values, the designers requested cambers throughout the fabrication process. The initial values reported at release were higher than anticipated. For example, the theoretical camber at release for beam 4 in span 4 was 2.5" and the reported value was 3.7", which is an increase of 46%. These increased cambers reduced the embedment height of shear reinforcing at the ends of the beams. In some instances, additional reinforcing was added before the deck pour to ensure minimum embedment into the deck slab. In other locations, pedestal elevations or the pier cap was lowered to accommodate the reported cambers. The nature of this Design-Build project allowed for the opportunity and flexibility to provide a solution during construction before beams were erected.

Beam erection began on a span by span basis in May 2013. The beams were transported from Vermont via flatbed trailers to a portion of the existing bridge. A 300-ton crane supported on a 54'x150'x10' barge in the river was used to erect the beams. (See Figure 5). For spans 2 thru 9, two beam lines were directly placed from the trailer and onto the pier caps. The other beam lines in spans 2 thru 9 needed to be supported on temporary falsework on the barge prior to final placement based on beam weights and crane pick radius limits. This general sequence required 8 crane picks per span and allowed the contractor to place a span (5 beams) between one to two days.


Figure 5: Beam erection from barge.

End spans 1 and 10 were not completed until months later based on time-dependent embankment consolidation and testing at Abutment #2 and access for construction at Abutment #1. A different erection method was required for spans 1 and 10 based on limited work area and draft for the large barge. The Contractor was able to place a smaller crane on both approaches and then use a sectional barge in the river to place the final 10 beams. One side of the beam was supported by the barge and the other side was supported on the approach. (See Figure 6). The barge was moved with winches to position the beams. Once the tide started to fall, the beam came to rest in its final location and the shoring was removed from the barge. The Contractor fabricated a steel pocket to guide the beam into its final position during winching operations and tidal fluctuations. (See Figure 7). Since the system was tide dependent, one beam was set per day. The final beam was erected in January 2014.

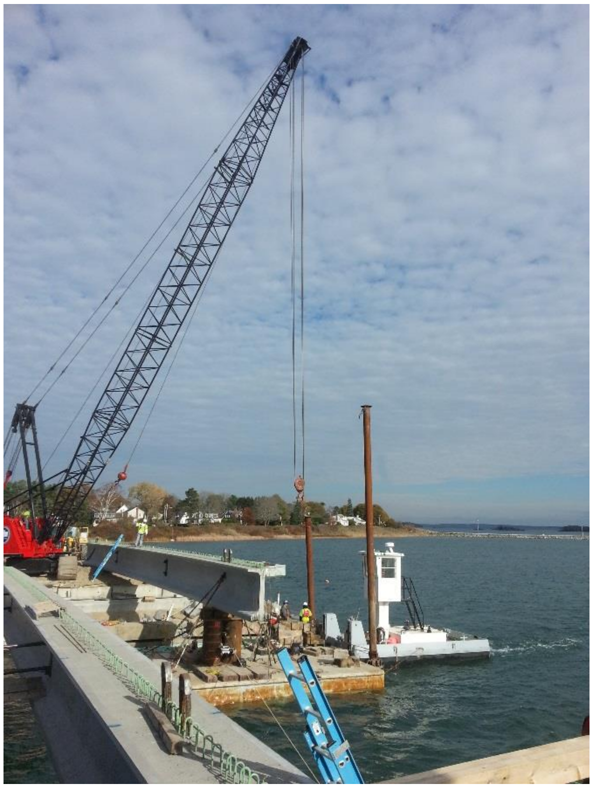

Figure 6: Beam erection at Span 10

Figure 7: Steel pocket guiding beam into place at Pier 9.

BEARING DESIGN

The use of integral piers simplifies the bearing design and details. The bearings on the integral piers were designed as unreinforced elastomeric pads to temporarily accommodate beam end rotations and the weight of the beams and deck before closure pours were completed. The use of unreinforced elastomeric pads instead of more costly reinforced elastomeric pads that would typically be used in non-integral construction, helped to make the structure economical.
Pier 5 bearings are expansion bearings that were designed as reinforced elastomeric pads with shear deformation to eliminate stainless steel plates and PTFE (polytetrafluoroethylene) surfaces.

INTEGRAL PIERS

The bridge is comprised of two continuous five-span superstructure units with fixed abutments, integral piers, and a traditional pier bent at the expansion joint near the middle of the bridge. The integral piers allow the superstructure and substructure to work together in response to limit state loadings to optimize superstructure and substructure design. This is especially beneficial in accommodating seismic and ice loadings. Each pier consists of six concrete filled steel pipe piles. The inner piles are plumb and the outer piles are battered slightly in the transverse direction to prevent significant service movements under wind loading and water pressure.

Pier	No. of Battered Piles	No. of Plumb Piles	Pipe Pile Diameter	Pipe Pile Thickness
Pier 1	2	4	30 in	0.75 in
Pier 2	2	4	30 in	0.75 in
Pier 3	2	4	30 in	0.75 in
Pier 4	2	4	30 in	0.75 in
Pier 5	2	4	30 in	0.75 in
Pier 6	2	4	24 in	0.75 in
Pier 7	2	4	24 in	0.75 in
Pier 8	2	4	24 in	0.75 in
Pier 9	2	4	24 in	0.75 in

Table 2: Pipe pile layout and sizes.

Piers 1 through 5 have heights of 22 to 36 feet above the mudline and piers 6 through 9 have heights of 10 to 15 feet above the mudline. To reduce tensile stresses in the superstructure under thermal movements, the shorter piers have 24x0.75 piles and the taller piers and expansion pier have larger 30x0.75 piles. The substructure was analyzed using *LARSA 4D* modeling software with the superstructure represented as a spline element. The pile bents were modeled as frames with depth to fixity based on equivalent fixed base cantilevers to match lateral load responses provided by the geotechnical engineer. This ensured pile moments would not be underestimated when subjected to lateral loads and thermal movements.

Structural analyses of the piles included moment magnification and were completed for the following conditions:

1. Driving and dynamic load testing with no concrete fill and a pinned head in the driving frame.
2. Concrete filled with fixed head and no section loss to the steel pipe.
3. Concrete filled with fixed head and section loss of 0.25” above mudline and 0.1” below mudline.

For conditions 2 and 3, the depth of fixity was based on a conservative approach using the second zero deflection point as determined by P-Delta analysis and soil pile interaction. This resulted in maximum unbraced lengths of 110 feet for the 30 inch diameter piles and 70 feet for 24 inch diameter piles. For condition 1, when piles were braced by a temporary driving frame and not subject to significant lateral loads, the effective length was based on the approximations provided in AASHTO C10.7.3.13.4. This resulted in maximum unbraced lengths of about 71 feet for the 30 inch diameter piles and 44 feet for the 24 inch diameter piles.

The nominal driving resistance required for the 30 inch and 24 inch piles was 1680 kips and 1501 kips respectively using a resistance factor of 0.65. To minimize the potential for pile damage during driving operations, the Design-Builder furnished steel pipe piles with a yield strength of 65 ksi instead of the 50 ksi minimum required by design as driving stresses were anticipated to be high.

The upper 30 feet of each pile is reinforced with 16 – No. 10 reinforcing steel bars bundled in pairs. The bars were equally spaced within No. 4 hoop bars spaced at 12 inches and extended 28 feet into the pile to achieve the composite moment strength where forces are greatest and piles are subject to ice loads. Without this reinforcement, a thicker pile section would be required for the upper section of the upstream piles where impact of the ice load is anticipated. All the piles are embedded into the pier caps two feet with the reinforcing cage extending 12 inches above the pile. (See Figure 8 & 9)

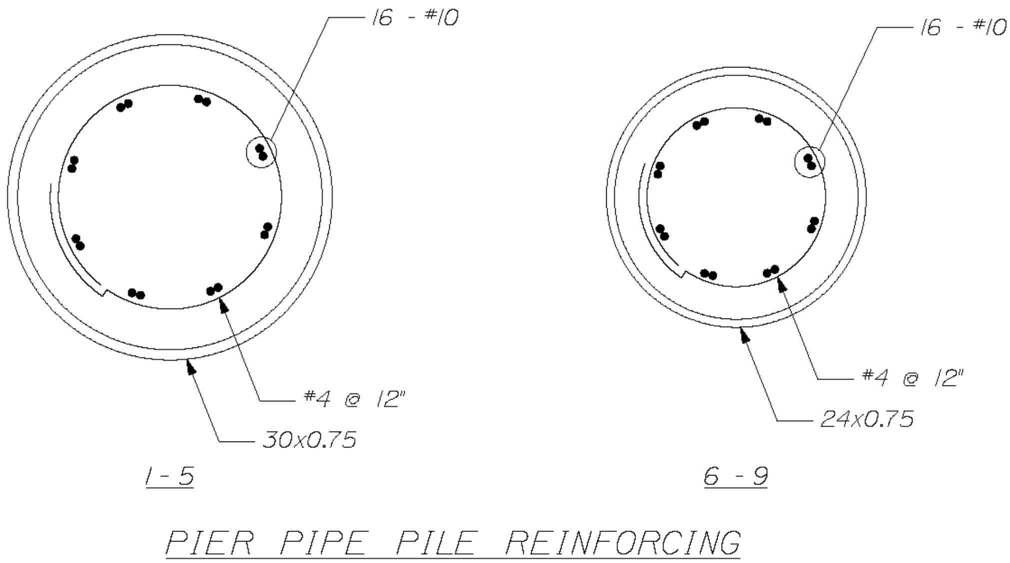

Figure 8: Pipe Pile Reinforcing (1 of 2)

Figure 9: Pipe Pile Reinforcing (2 of 2)

During the construction stage, the pier caps at the integral piers were designed for dead load and construction live load with a design depth based on the pier cap depth only. Once the integral closure pour was in place, the depth for the reinforced concrete design was increased to the top of the deck. This large ‘d’ distance was able to accommodate the high moments associated with the integral connection. Pier 5, the only non-integral pier, required a much wider pier cap to resist loads since it didn't have the benefit of larger depth at the final condition.

The integral connection in the pier cap was detailed with flush mechanical connectors rather than extending rebar into the closure pour. This was done to allow the Contractor the option of sliding the superstructure into place.

CONSTRUCTION CHALLENGES

INTEGRAL PIER DIAPHRAGM POURS

Since the integral caps and deck were heavily reinforced, with spacing as small as three inches in some locations, pouring concrete was especially difficult. To ensure consolidation, several bars in the top of the deck were removed to allow the 4,000 psi, Class A concrete access into the pier diaphragms and then secured into final position as the concrete reached the level of the deck. Figure 10 shows the typical deck reinforcing and tight constraints due to tight spacing.

Figure 10: Typical deck reinforcing layout.

PILE INSTALLATION

Although the alignment of the new bridge was off-line of the existing bridge, there were remnants of timber pilings from an earlier bridge that would likely interfere with placement of the new piers. Fortunately, the timber pile remnants were easy to extract as there was enough pile projecting above the mudline and they were not very long. If the piles could not be removed, the new pile cap was designed to allow a transverse pile shift of about two feet.

During construction, a few of the pile tips broke during the pile driving and soil filled the bottom few feet of pile. After failed attempts to clean and remove the sediment, the pile was reanalyzed. Instead of determining the depth of fixity based on the initial approach of using the second zero deflection point as determined by P-Delta analysis and soil pile interaction, an average of the first and second point of zero deflection was considered. Based on the actual pipe pile material strength (higher than design value of 50ksi) and a less conservative depth to fixity, the steel pile itself had sufficient capacity.

In addition to issues with the pile tips under high driving stresses, one of the piles was not able to meet the required driving criteria using a resistance factor of 0.65. This was remedied by performing dynamic testing for all of the piles in the pier bent to allow a higher resistance factor of 0.75 thereby reducing the driving criteria. (See Figure 11 for typical pile installation.)

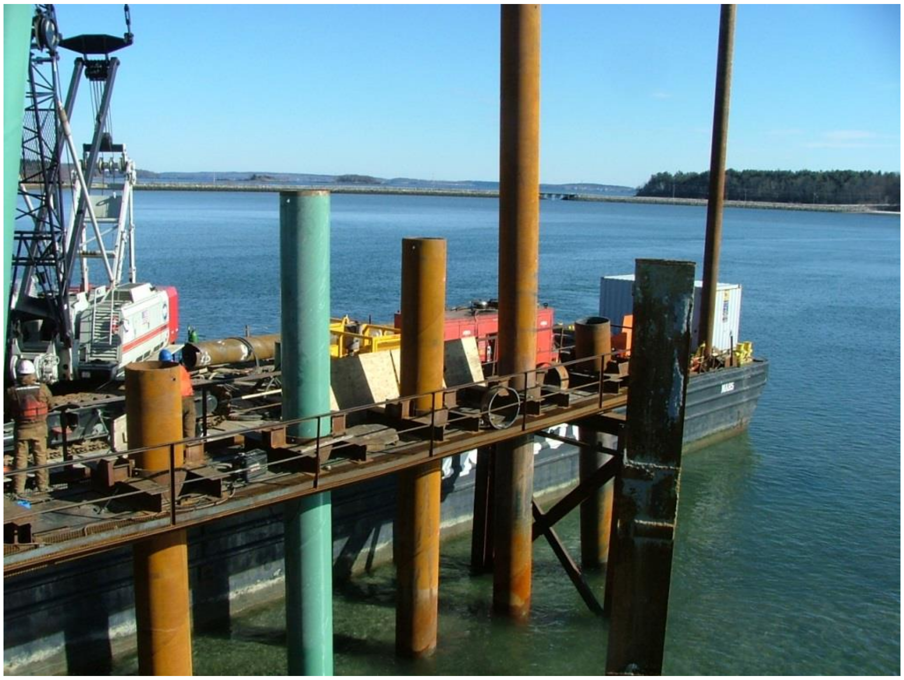

Figure 11: Pile installation

CONCLUSION

The Design-Build price proposal was $23.5 million. The total project cost estimated by MaineDOT was between $30 and $35 million. Other price proposals for this project ranged from $25.7 million to $40.4 million. Estimated savings with this innovative design is $2 million based on the difference in the next lowest price proposal using a conventional superstructure and pier bent configuration. The innovative design of multiple integral piers with precast/prestressed girders at the Martin's Point Bridge proved to be an economical structure for the Maine Department of Transportation and may serve as a model for future projects.