FEASIBILITY STUDY OF BRIDGE REPAIR AND REPLACEMENT ALTERNATIVES
LAFAYETTE AVENUE (M-13/M-84)
OVER THE EAST CHANNEL OF THE SAGINAW RIVER

PREPARED FOR:
MICHIGAN
DEPARTMENT OF TRANSPORTATION

JN: 117082

STRUCTURE NO: 586, BRIDGE ID: B01-09032

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1 EXECUTIVE SUMMARY

The Lafayette Avenue Bridge with bascule span crosses the east channel of the Saginaw River on the south side of Bay City. Its movable span opens frequently for commercial and recreational vessels during the navigation season.

The three-span structure is comprised of a double leaf rolling lift bascule span flanked on both ends by single steel plate girder approach spans. The structure has a total length of 456 feet. It has a 30’-0” roadway width between face-to-face of curbs and carries one traffic lane in each direction. There are 5’-0” wide sidewalks on both sides of the roadway.

The bridge was originally constructed in 1938. Its bascule and approach span superstructures were replaced in 1987. Since its replacement, the bascule span has exhibited serious problems with the curved steel tread plates that are attached to the heel portions of its bascule girders and the mating flat track plates that are mounted into the piers. These problems have resulted in numerous repair efforts and continue to require frequent attention. This report documents a study of alternatives for rehabilitating and completely replacing the bridge.

As part of the rehabilitation alternative, extensive work would need to be performed on the heel portions of the bascule superstructure to help ensure continued reliable operation for an extended period of time. The major element of that work would be complete removal of the steel tread plate and track plate assemblies and replacement of them with more robust higher strength ones, attached with improved connection details. That work would require temporarily jacking and shoring each of the bascule leaves and their associated counterweights.

Several alternative structural configurations were investigated for the bridge replacement alternative. A double leaf rolling lift bascule structure flanked on each end by single precast prestressed concrete girder approach spans was determined to be the most effective one. The replacement alternative is based on a new structure being constructed on the same alignment as the existing bridge, which would limit right-of-way impacts but require a multi-year closure with detour for bridge users.

Key criteria were formulated to compare the two alternatives. Because of the greater extent of work associated with a bridge replacement, that alternative would have a substantially higher initial construction cost compared to a bridge rehabilitation. Based on a 90-year life and including both the estimated cost of an initial construction project and costs for project future needs, life cycle cost analysis computations indicate that the bridge replacement alternative would also have a greater net present value.

Replacing the bridge with an entirely new structure would provide a greater likelihood for long term reliability compared to rehabilitating its 80 year old substructures and 30 year old superstructure, mechanical and electrical systems. Rehabilitating the bridge would also have the risk of a greater potential to encounter unforeseen issues during construction. Rehabilitation
with the in-place replacement of the tread and track plate assemblies would involve highly complex construction techniques compared to the relatively straightforward construction of a full bridge replacement.
2 INTRODUCTION

This purpose of this structure feasibility study was to evaluate and compare rehabilitation and replacement alternatives for the Lafayette Avenue Bridge (Structure No: 586, Bridge ID: B01-09032) over the east channel of the Saginaw River in Bay City, MI.

The main span is a double leaf rolling lift bascule bridge with a Scherzer style center lock. The bridge is flanked on both ends by single steel plate girder approach spans. The bridge was originally built in 1938 and has since undergone several rehabilitations.

2.1 BACKGROUND

The bridge is owned, operated and maintained by the Michigan Department of Transportation (MDOT). The main channel at the bridge is under the jurisdiction of the United States Coast Guard. The waterway supports large commercial vessels that operate on the Great Lakes System, as well as United States Coast Guard vessels, and recreational navigation.

Bridge opening logs were reviewed for the time period between April 2011 and December 2012. On average, there were 443 bridge openings per year excluding openings for maintenance, testing or other non-vessel related work. Peak use of the channel occurs between the months of April and October. The bridge is generally not operated during the winter months between the beginning of January and the end of March.

2.1.1 Location

The bridge is located approximately 2.5 miles east of I-75 in Bay City, Michigan. It carries Lafayette Ave. (M13 & M84) over the east channel of the Saginaw River. The east channel joins the west channel just north of the crossing and then the river flows north for 6.8 miles before it outlets into Saginaw Bay on Lake Huron. There is a fixed structure located to the west of the bascule bridge that carries Lafayette Avenue over the west channel. To the north (downstream) of the Lafayette Avenue crossing there are three other highway bascule bridges - the Veterans Memorial Bridge owned by MDOT and the Liberty and Independence Bridges owned by Bay City.
2.1.2 History of Bridge

The structure was originally constructed in 1938. The bridge underwent a major rehabilitation in 1987 with work that encompassed a complete bascule span and approach span superstructure replacement, pier and abutment rehabilitation, implementation of scour countermeasures, and construction of additional timber dolphins. Since the rehabilitation in 1987, the bridge has experienced numerous problems with the bascule girders’ tread plates and mating track plates upon which they roll. Multiple repairs have been performed on them in an attempt to address the problem.

In 2000, a mechanical and electrical rehabilitation was performed when a new programmable controller (PLC) was installed to replace the PLC installed in the 1987 rehabilitation. The rehabilitation also included console modifications and limit switch replacements. Mechanical work consisted of replacing the rotary cam limit switch and resolver along with the mating couplings and drive train. The live load shoes and center locks were shimmed. The motor brake, bellows and limit switch supports were replaced on the rear lock actuators.

A 2006 rehabilitation provided for the replacement of expansion joints and strip seals, an epoxy overlay of the bascule span, repairs to slope protection, a partial cleaning and painting of steel on all spans, as well as other miscellaneous structural work.

2.1.3 Description of Existing Structural System

Exhibit D-1 shows a plan and elevation of the existing structure. For the purposes of this report, the abutments are referred to as east abutment and west abutment. The bascule piers are referred to as east pier and west pier.
The three span bridge is on a vertical curve. It has a total length of 451’-6” from centerline of the west abutment to centerline of the east abutment. The west approach span has a length of 96’-10” from bearing-to-bearing and the east approach span has a length of 107’-8” from bearing-to-bearing. The bascule span has a length of 185’-0” between first positions of roll. The bascule span provides a vertical clearance of approximately 20 feet when closed. It provides a clear navigation channel width of 150 feet. The bridge carries two lanes of traffic with a clear roadway width of 30 feet between curbs. Sidewalks widths of 5’-0” accommodate pedestrians on each side of the roadway.

The approach span superstructures consist of five 60-inch welded plate girders. The approach span girders are supported by expansion bearings at the east and west abutments and by fixed bearings at both bascule piers.

The movable span is a double leaf, rolling-lift bascule structure. Each leaf of the bascule span includes two variable depth welded plate girders (bascule girders) that support a system of floorbeams and stringers. When the bridge opens and closes, curved steel tread plates attached to the bottom of the two bascule girders of each leaf roll on steel track plates mounted to the piers. Each bascule leaf is balanced by a concrete counterweight with support framing that spans between the heel section of each bascule girder. A girder that spans transversely between the two rear rack columns provides an uplift reaction support for the bascule girders to prevent the leaves from rolling forward under live load. The control tower for the movable span operations is situated on the north side of the east bascule pier (See Photo S-4).

The substructures consists of two bascule piers and an east and west abutment. Deep timber pile foundations support the substructures. The substructures have had concrete patching and crack sealing repairs performed on them in previous rehabilitations with their primary structural reinforced concrete elements remaining from original construction. The concrete control tower was completely reconstructed on the east bascule pier during the 1987 rehabilitation project.

The bridge is considered scour-critical. As part of the 1988 rehabilitation, scour countermeasures were provided to address areas where scour had undermined and exposed the supporting timber piles at the upstream end of both bascule piers. That work consisted of installing grout-filled bags in scoured areas and placing grout backfill beneath the portions of undermined foundation seals.

A hydrographic survey of the bridge was performed in 2013 after high water conditions occurred during the spring of that year on the Saginaw River. That survey identified two new large scour holes that had developed on the downstream side of the bascule span. One scour hole was found approximately 45 feet northwest of the east bascule pier’s northwest corner. Another scour hole was found approximately 44 feet east of the west bascule pier’s northeast corner.
2.1.4 Description of Existing Mechanical System

Each of the two leaves is operated by separate machinery systems located in machinery rooms on each of the movable leaves. This machinery is located under the concrete deck of the roadway on the heel portion of each leaf. The machinery layout is symmetric about the centerline of the bridge. Two 30-horsepower electric motors drive the machinery system of each leaf. That system consists of one primary enclosed differential speed reducer with output shafts driving two secondary enclosed speed reducers located either side of the primary reducer. The secondary speed reducers drive output shafts passing through the bascule girders with pinions on their ends that engage fixed racks attached to stationary support frames on the outboard side of the bascule girders. Each bascule leaf has one motor brake and one machinery brake. Though the two brakes for each leaf are identical and each one acts on one the primary reducer’s two input shafts, they provide separate functions. The motor brake is used to stop the leaf when it is moving and the machinery brake is used to hold the leaf in a fixed position when it is not moving. Figure 2-2 shows the general configuration of the bridge’s mechanical system.

![Figure 2-2: Layout of Existing Mechanical System](image)

2.1.5 Description of Existing Electrical System

The electrical system at the bridge provides power and control for operation of the movable span, traffic warning devices, and channel navigation signals. The system is controlled with a programmable logic controller (PLC) based design. Two 30 horsepower shunt wound DC motors are speed controlled by a single variable speed DC drive. Each leaf has a backup variable speed DC drive, one of which does not work.

The bridge’s electrical system is powered from the local utility with a 3-phase, 480-volt, 600-amp service. In the event of power failure, backup power can be provided by a 270 KW Caterpillar emergency generator that is housed in the east bascule pier. Submarine cables carry power and control under the waterway from the east bascule pier to the west bascule pier for...
operating the equipment on the opposite leaf. The electrical system was replaced in the 1987 superstructure replacement project. In the 2000 rehabilitation project, a new PLC system was installed to replace the old one.

2.2 PROJECT STUDY APPROACH

2.2.1 Review of Available Information

An in-depth inspection of the structural, mechanical and electrical systems of the bridge was performed between September 26 and September 30 of 2011. The findings from that inspection are documented in a detailed inspection report dated February 2012 (2012 In-Depth Inspection, Hardesty & Hanover).

Additional reference documents utilized for this study include:

- Lafayette Avenue Original Design Plans, 1938
- Lafayette Avenue Rehabilitation Plans, 1987 (Superstructure Replacement)
- Lafayette Avenue Rehabilitation Plans, 2000 (Electrical and Mechanical)
- Lafayette Avenue Rehabilitation Plans, 2006 (Misc. Structural)
- Maintenance records
- 2010 Underwater Inspection Report and Scour Action Plan
- Survey monitoring data for the northwest track and tread plates
- Load Ratings
- Span Balance Calculations
- Bridge Inspection Report, 1999, Lichtenstein Consulting Engineers

2.2.2 Supplemental Site Visits and Inspections

At the initiation of this study a two day site visit and walk-through inspection of the bridge’s structural, mechanical and electrical systems was made on July 17th and 18th, 2013. As part of this site visit, a strategizing meeting was conducted to consider potential details for bridge rehabilitation and to confirm major parameters for a replacement bridge. During the second day of that inspection, drift tests were performed to observe the balance condition of each leaf.

A follow-up inspection of the underside of the bridge’s superstructure was performed on August 13, 2013 with the use of an under-bridge access unit. As part of that inspection, the bascule piers and navigational protection systems were inspected with up close access provided by boat.

Photographs from the July and August inspections are included in Appendix E.

2.2.3 Development and Evaluation of Alternatives

Site visits, supplemental bridge inspections and available reference information were used in developing concepts and details for two primary alternatives - a major bridge rehabilitation and a complete bridge replacement. Work scope and details were developed and evaluated to enable
comparison of these alternatives. Although initial project construction cost was an important consideration, additional factors for consideration include:

- Life Cycle Costs
- Functionality
- Long Term Reliability
- Risk
- Constructability
- Construction Disruption
3 DESIGN CRITERIA & ASSUMPTIONS

3.1 FHWA and MDOT Requirements

3.1.1 Traffic

Lafayette Avenue is functionally classified as an urban other principal arterial trunkline. According to Michigan.gov, the ADT was determined to be 21,300 vehicles per day (vpd) in 2012, with a commercial traffic of 6%, projecting to 25,100 vpd in 2025, as recorded in the most recent Structure Inventory and Appraisal database entry.

3.1.2 Geometry

The roadway width on the existing bridge is 30’-0” from curb to curb and carries two lanes of traffic. There are 5’-0” wide sidewalks on each side of the roadway. The design speed for this structure is 35 mph. The bridge deck has a cross slope of 1.5% which is less than the desirable 2% slope. The existing bridge rail is not classified by MDOT as crashworthy.

The existing profile of the structure is on a vertical crest curve. The curve length is 185 feet which is greater than the required stopping sight distance of 174 feet per MDOT Sight Distance Guidelines.

3.1.2.1 Rehabilitation Alternative

For a rehabilitation alternative, the existing traffic lane and sidewalk configurations would remain unchanged. Their existing widths meet MDOT standards and therefore would be sufficient. A bridge rehabilitation would incorporate a new crashworthy railing system on the approach spans and the bascule span.

3.1.2.2 Replacement Alternative

The MDOT Roadway Design Manual Chapter 3 Appendix 3A and Chapter 6 of the MDOT Bridge Design Guide (MBDG) were used to determine geometric design guidelines. The MBDG states that for new construction, the roadway width of trunkline bridges with a maximum posted speed of 40 mph shall be equal to the minimum lane width times the number of lanes plus 2’-0” shy distance to curbs. The proposed replacement alternative would provide three 12’-0” lanes with 2’-0” shy distances to curbs. This would result in a total curb-to-curb width of 40’-0”. Sidewalks 5’-0” wide would be located on both sides of the roadway. The replacement configuration would bring all geometric design criteria up to current standards as well as add a lane for an increased level of service and better access for future structure maintenance. The new bridge railing system would meet current MDOT standards as well. The replacement alternative assumes the new structure would be built to the same horizontal and vertical alignment as the existing structure. Preliminary contact with the U.S. Coast Guard (USCG) was
made concerning the vertical and horizontal clearances. It was conveyed that the current USCG listing of 150 feet for horizontal clearance and 20 feet for vertical clearance would be considered minimums for a new bridge and that the bridge permitting process would develop the final requirements. Lafayette Avenue Bridge currently has the lowest vertical clearance in the area. The downstream Veterans Memorial Bridge has the next lowest clearance at 23 feet. For the purposes of this report, the limits of work for the replacement alternative include 40 feet of roadway construction beyond each of the new abutments.

### 3.1.3 American Disability Association (ADA) Requirements

Sidewalk ramps and crosswalks on and adjacent to the bridge require proper cross slopes, tapers and crossing equipment for proper accessibility. Sidewalks must maintain a cross-slope not exceeding 2%, longitudinal grades exceeding 5% must follow the requirements for a ramp. If sidewalk width is less than 60 inches, a passing space must be provided at 200 foot intervals.

MDOT standard curb ramps and bridge railings are ADA compliant.

### 3.2 Federal Agencies

The existing bridge provides a navigation channel that is 150'-0” wide between the faces of its navigation fenders. Further coordination with the USCG and results from a full permit process that includes comments from stakeholders, general public and other governmental agencies will be required during design and construction. The bridge replacement alternate would require Section 404 of the Clean Water Act permit authorization administered by the U.S. Army Corps of Engineers due to substructure work in the channel bottom. Construction or modification of a bridge in the navigable Saginaw River will require a Coast Guard Permit.

### 3.3 Department of Natural Resources and Environment

A permit must be obtained from the Michigan Department of Natural Resources and Environment for the crossing of an inland stream by a bridge. Proper measures will be required to contain any river bottom sediments disturbed during construction. Any contaminated river bottom sediment excavation encountered during construction would require special handling and disposal. The Saginaw River is not defined as a natural, or wild and scenic river as detailed in the MDOT Bridge Design Manual Appendix 14.07.01.
4 BRIDGE REHABILITATION

Long-term rehabilitation recommendations would maintain and extend the reliable service of the bridge for an extended period. It is expected that implementation of the recommended long-term rehabilitation actions would extend the life of the bridge for approximately 50 years, which would mark the end of its useful life. This assumption is also carried forward in life cycle cost analysis provided in a later section. Throughout this extended life of the rehabilitated structure, routine maintenance, deck repairs, painting and electrical and mechanical rehabilitations would still be periodically required. However, it would eliminate the need for another major structural rehabilitation and eliminate the continuous maintenance problems associated with the failing track and tread elements. At the end of the extended useful life of the bridge, a complete bridge replacement would be required.

4.1 STRUCTURAL SYSTEM RECOMMENDATIONS

The following subsections provide descriptions and recommendations for rehabilitation of the structural system of the bridge including its fixed approach spans, bascule span, substructures, navigational protection system, and operator house.

4.1.1 Approach Spans

There is one approach span flanking each end of the bascule span. Both approach spans consist of reinforced concrete decks supported by five 60-inch deep web plate girders spaced at 9’-6”. Raised sidewalk slabs poured on top of the bridge deck, flank each side of the roadway and have steel bridge railing mounted to them.

4.1.1.1 Approach Span Deck and Sidewalks

The approach spans were replaced as a part of the 1987 rehabilitation project. The decks are 9-inches thick, epoxy-coated steel reinforced, and are supported on steel plate girders with shear studs for composite action. The 2006 rehabilitation provided for patching of the deck on the east and west approach spans. Generally, both approach spans are in the same condition and exhibit sidewalk map cracking, transverse cracking on the underside of the deck, and localized spalls on the curbs and edges of the concrete deck.

Overall, the surfaces of the approach spans decks are in good condition. During the follow-up inspection, the spans were sounded with a chain and only a few locations with hollow sounding delamination were identified. This confirmed the 2012 detailed inspection report that indicated that less than 1% of the total deck area was delaminated.

The underside of the approach span decks are generally in good condition. Hairline transverse cracks were consistent throughout the underside of both approach spans (See Photo S-16). The existing plans show that transverse reinforcement in the bottom of the deck is spaced at 6 1/2”
with only 7/8” clear distance. In some locations, it appears that the plan reinforcement spacing correlates to field measured transverse crack spacing. The cracks are leaching in some locations.

While only minimal delamination was detected in approach span sidewalks, heavy map cracking is prominent throughout the north and south sidewalks of both approach spans (See Photo S-6). The approach span curbs show general wear with a spall located on the north curb of the east approach. Other curb areas are showing signs of similar potential problems. The expansion joints are currently functioning properly.

By 2016, the approach span decks and sidewalks will be 26 years old. Based on their existing condition, there will still be several years of remaining life. As part of a bridge rehabilitation alternative, an epoxy overlay should be applied to the approach deck roadway surface. Considering the extensive map cracking and continued spalling of curbs, the sidewalks should be removed and replaced. Any spalls on the deck fascia coping should be patched as well (See Photo S-12). Expansion joints will be 10 years old and it is recommended that while epoxy overlays are being applied to the approach decks, the expansion joints should be removed and replaced.

4.1.1.2 Approach Span Railing

The approach span and bascule span both utilize a four-tube steel railing system that terminates at concrete parapets at abutments and piers. The railing throughout the structure is in good condition with no signs of impact damage. The 2006 rehabilitation plans indicate that the concrete parapets were to be repaired or replaced and field visits confirmed that repairs have been made in these locations. However, map cracking is prevalent in many of the repair areas. Although unsightly, the repair areas appear to be sound (See Photo S-14).

The existing four-tube steel railing does not conform to the current MDOT four-tube railing per Standard Plan B-26 and therefore does not meet crash testing criteria. More importantly, it also does not meet the 6” sphere pass through requirements of modern railing systems. A rehabilitation project would incorporate the removal and replacement of approach span sidewalks. Based on non-problematic crash history at the bridge and the relatively low posted speed limit it is possible that the existing rail could be reused. However, it would need to be modified for the 6” sphere pass through criteria. Considering this it is recommended to bring all the railings up to current standards, a new four-tube, sidewalk-mounted railing that is approved by MDOT as crash tested should be installed in both the approach spans and the bascule span.

4.1.1.3 Approach Span Structural Framing System

The steel girders and supporting bearings of the approach spans are in good condition and can continue to provide reliable service for an extended period of time.

MDOT was consulted in estimating the remaining life of the paint system. A partial painting was performed in 2006 for both approach spans and the bascule span structural steel using MDOT’s current 3-coat system (organic zinc primer, epoxy intermediate & urethane top coat).
Given the existing conditions, it is estimated that a full superstructure abrasive blast clean and re-painting is not required again for approximately 25 to 30 years. A partial painting (zone painting) is recommended again for the rehabilitation project since many of the painted areas will be impacted by steel repair work during the work.

4.1.1.4 Abutments

The east and west abutments are generally in good condition. They do display some localized map cracking as well as vertical cracking (See Photos S-35 and S-36). For a rehabilitation project, it is recommended to seal map cracked areas and perform concrete repairs to any areas that have pop outs or have delamination.

4.1.2 Bascule Span

The bascule span consists of two rolling-lift bascule leaves over the navigation channel with a total clear width of 150’-0” between fenders. Each leaf of the movable span has a front section that overhangs the channel and consists of two bascule girders that support a system of floorbeams and stringers. The front-arm framing supports a concrete half-filled steel grid deck. Jaw and diaphragm type center locks at mid-span serve to transfer live loads between movable leaves.

The heel section of each leaf consists of a concrete counterweight and structural framing system that connects the counterweight to the bascule girder tail sections and supports a concrete bridge deck. A fixed live load uplift girder framing between the rear columns of the rack frames provides a reaction point for the bascule girder tails to prevent the bridge from rolling forward under live load highway traffic.

4.1.2.1 Steel Grid Deck and Sidewalks

The bascule leaves support a latex modified concrete half-filled steel grid deck from the center breaks to a point 4’-9 1/2” forward of the centerline of the rear break. The steel grid deck is in good condition. The thin epoxy overlay on the surface is worn, and the steel grid is beginning to reflect through the surface. The underside of the concrete fill is not visible due to metal form pans but shows nothing that would indicate problematic issues. The half-filled grid deck is original to the 1987 superstructure replacement project, making it 26 years old.

The bascule span sidewalks consist of latex modified concrete filled steel 2” “T” grid that is supported by stringers and cantilever brackets. There is an epoxy overlay on the sidewalk that is degrading and the steel grating is reflecting through to the surface (See Photo S-5). From below, the grid appears to be in good condition.

In 2016, the steel grid bridge deck and bascule span sidewalks will be close to 30 years old. For a bridge rehabilitation the epoxy overlays on both the roadway and sidewalks should be removed and replaced.
4.1.2.2 Concrete Deck and Sidewalks on Heel Portion

The heel portion of the bridge deck consists of a concrete half-filled steel grid deck that is a continuation from the front-arm of the bascule superstructure to a point 4’-9 1/2” from the centerline of the rear breaks. At the termination of the steel grid, a reinforced concrete slab is directly supported on top of the counterweight (See Photo S-11). A neoprene block is anchored to the end of the pour and serves as the movable side of the rear break by butting up to the fixed steel plate on the approach. In 2006, the neoprene blocks were replaced, and the concrete slab was chipped down and overlaid with concrete.

The concrete deck over the counterweights is in a similar condition as the approach span concrete decks with some minor defects. During the follow-up inspection, a chain drag identified a few localized delamination locations.

The sidewalks in this region widen out to cover the footprint of the bascule pier. These sidewalks show wear and cracking, especially in the region of the longitudinal joints.

As a part of the rehabilitation alternative, it is recommended that the concrete deck portion over the counterweight have an epoxy overlay applied consistent with the work recommended for the approach span decks. Sidewalks in this area should be removed and replaced at the same time the approach span sidewalks are removed and replaced.

4.1.2.3 Bascule Span Railing

The bridge railing on the bascule span is the same four-tube section used on the approach spans (See Section 4.1.1.2). Railing posts are bolted directly to steel bolsters on top of the cantilevered steel sidewalk brackets. As discussed in the earlier section it is recommended that the railing be replaced as part of a rehabilitation project.

4.1.2.4 Roadway and Sidewalk Breaks

The center, rear, and longitudinal breaks of the bascule span deck separate the movable portions from each other and from the adjacent fixed portions. The breaks consist of rolled steel plates and shapes, as well as some neoprene elements.

The roadway and sidewalk center breaks, which are located at the center of the bascule span, separate the two movable leaves. The center breaks for the sidewalks are in good condition. However, the teeth on the east leaf of the north sidewalk are approximately 5/8” higher than the teeth on the west leaf. The center break at the roadway has tooth clearances smaller than indicated on the 1987 rehabilitation plans. After accounting for temperature differences between inspection conditions and plan dimensions, it was determined the breaks are closer together than indicated on the rehabilitation plans by approximately 1 3/4” at the south end of the break and 1 1/2” at the north end. This condition could be the result of the original installation not meeting plan dimensions, pack rust between the vertical break plates and connecting framing, or a combination of both. There are no indications that this discrepancy in break clearance is a result
of the bascule leaves or piers moving toward one another. MDOT has trimmed the fingers of the roadway center break by flame cutting because they were binding during hot weather conditions.

The roadway center break should be replaced as part of a bridge rehabilitation. An in-kind replacement with shorter teeth may be the best means of doing this as it would avoid replacement or modification of the supporting channels, cantilever brackets and the abutting concrete filled grid deck, all of which are still in sound condition.

The sidewalk center break should also be replaced as part of a bridge rehabilitation. The supporting 8”x8”x3/4” angles could be kept and just the break plate itself replaced with one not having fingers. That arrangement with a narrow transverse open joint is typical of what is often provided for sidewalk center breaks on bascule bridges. Providing a slip resistant coating on the new break plate surface would also be desirable.

The rear breaks for the roadway and sidewalk are located at the heel portion of the bascule leaves. The fixed rear break plate is in good condition (See Photo S-11). There is a neoprene block at the rear break that provides a soft transition across the break in the floor. The block creates a seal at the fixed steel plate that discourages water from flowing through the joint. The neoprene blocks would need to be removed in order to jack the leaves for recommended track and tread work. It is recommended that the neoprene blocks at the rear break be replaced as a part of the rehabilitation alternative.

The longitudinal breaks are the joints that separate the movable portion of the deck system supported by the bascule girders from the adjacent fixed sidewalk concrete decks, which are supported by the rack girders at the bascule pier walls. These joints are in poor condition. A “doubled-over” woven neoprene reinforced seal is incorporated to close the gap and deter water from leaking down into the pier. The seal slides along the fixed steel portion of the break as the bridge operates. The seals are worn out in all locations and have never been replaced. The concrete surrounding the joints have map cracking and spalls (See Photo S-8). Additionally, what has been described in previous reports as rubbing of the counterweight against the bascule pier walls in the NW and SE quadrants during operation is actually the longitudinal break plate rubbing (See Photo S-29).

As a part of a bridge rehabilitation project, the longitudinal joints should be replaced along with the adjacent concrete in which the components are embedded. We typically use a replaceable neoprene extrusion that is clamped in place by steel parts. We also provide a slight skew to the joint such that fits tight with the bridge in the closed position, but tends to separate from the fixed steel elements as the bridge rolls back during opening. This reduces the amount of wear on the neoprene.

4.1.2.5 Bascule Girders

There are two built-up, welded plate, bascule girders that support each leaf of the structure. The heel section of each girder is curved with two, three-inch curved plates bolted on to form the
tread. Behind the first position of roll, the upper half of each girder extends to connect to the counterweight. Beyond the counterweight, the girder extends to provide a surface for uplift support. The front section of the girders supports the floorbeams, stringers, and grid deck of the bascule span.

The bascule girders are in poor condition. The primary deficiencies are associated with the problematic tread plates. This is discussed in-depth in the section “Tracks and Treads”. The bascule girders also showed some areas of localized paint chipping and surface rust.

The 2012 Detailed Inspection Report noted that the live load bearings at the heel of the bascule girders were in good condition with surface rust and minor pack rust on all the bearings. The report stated that all bearings had a tight fit with minimal gaps in the closed position. We observed a similar condition.

The paint condition of the bascule girders is summarized in the section “Approach Span Structural Framing System”. That section explains that the paint system should not need a full blast clean and painting for approximately 25 to 30 years.

Spot painting should be done during a proposed rehabilitation alternative and during subsequent bridge projects throughout the useful life of the structure. While live load bearings appear to have a tight fit, shimming would be required as a result of the work needed at the center locks.

4.1.2.6 Front Arm Structural Framing

The bascule structural front arm framing system out over the navigation channel of each leaf consists of six transverse floorbeams that support seven rows of interior longitudinal stringers, curb stringers, cantilever sidewalk brackets, sidewalk stringers, and lateral bracing. The half-filled steel roadway grid deck is welded to the top flanges of the stringers.

Overall, the only deficiencies of the front arm structural framing are in the floorbeam end copes. In many of the floorbeams, cracks have developed in the web at the members’ connection to the bascule girder. The cracks extend from the coped edge at the top of the connection, and typically end where crack relief holes have been drilled. There are 14 total crack locations, and in three of those locations, the crack appeared to extended beyond the drilled relief hole and another relief hole was drilled to stop the crack.

The 2012 detailed inspection noted that no crack propagation was observed beyond the drilled relief holes. During the URS follow-up inspection in August of 2013, two locations were observed where new cracks extended beyond drilled relief holes. These two locations were at the south side of floorbeam 4 on the west leaf, and the north side of floorbeam 4 on the east leaf.

Figure 4-1: Floorbeam Crack
The cracking appears to have occurred because of out-of-plane bending of the web when the structure is in the open position. Structural WT sections serve as lateral bracing for the sidewalk framing that overhangs the bascule girders. These WTs terminate at a bolstered connection block on top of the floorbeams just above the coped areas. The sidewalk curb stringer is also mounted on this bolster. When the leaf opens, the weight of the sidewalk, sidewalk stringers, cantilever brackets and railings tend to transfer load through this lateral bracing as a tension force that pulls on the connection bolster to the top flange of the floorbeam. The top flange is essentially working back and forth and the coped edge of the floorbeam web is vulnerable to crack formation. The bending impacts can be magnified when the moving structure stops abruptly and rocks back and forth.

As a part of the bridge rehabilitation alternative, this issue should be fully addressed. Each hole should be examined for any new cracks that have propagated through them using dye penetrant. New relief holes should be drilled or existing ones enlarged to blunt the tip of any cracks found. The cracked areas should then be cleaned and painted. After painting, these crack locations should be retrofitted. Structural steel strengthening elements should be drilled and bolted in place to bridge across the problematic areas to provide adequate strength and stiffness to the floorbeam connection. Figure 4-2 shows a schematic of a suggested repair. A bolted angle shaped weldment could be placed across the cracked area and tied into the top flange and the floorbeam end connection on one side. The relief holes would be filled with caulk to seal out

Figure 4-2: Floor Beam Retrofit Schematic
moisture. This would stiffen the member in the weak direction and provide continued visual access to the crack from the opposite side during future inspections.

In addition to the floorbeam retrofit, spot painting of the entire front-end structural framing system should be done during a rehabilitation and during subsequent bridge projects throughout the useful life of the structure.

4.1.2.7 Heel Section Structural Framing

The heel section structural framing system consists of top and bottom counterweight bracing, vertical counterweight trusses, machinery support framing, and the fixed live load uplift girders. The counterweight bracing, trusses and machinery support framing had no significant deficiencies. The live load uplift girders have cracks in the coped portion of the webs where the bottom flange terminates at the end connections (See Figure 4-3 and Photo S-20). Four cracked locations were identified, one at each end of both uplift girders.

The live load uplift girders span between the rear columns of the rack frames and provide a reaction point for the tails of the bascule girders when traffic is on the bridge, to keep the girders from rolling forward. When the bridge is in a closed position, the tails of the bascule girders bump up flat across the bottom flange of the uplift girder at the reaction point. The web cracking at the cope is likely the result of non-uniform loading on the uplift girder flange. If the contact surface at the uplift point is not perfectly parallel, the flange will tend to rotate under load and put high stress concentrations in the web cope.

Similar to the bascule span floorbeams, relief holes have been drilled to prevent crack propagation. The August 2013 follow-up inspection showed that conditions at the uplift girders have worsened since the 2012 detailed inspection report. It was also observed that more relief holes have been drilled to combat the ongoing crack propagation. The 2012 detailed inspection report noted that the west uplift girder had a crack that stopped at the second of two drilled relief holes on the north side of the member. The report also noted a crack with one relief hole at both ends of the east uplift girder. In all three locations, the cracks have propagated and another relief hole has been drilled and is currently successful in arresting the cracks. In the fourth location, where the 2012 report noted a crack was forming, the crack is now evident and a relief hole has been drilled to prevent further propagation.

The method of stopping crack propagation by means of drilling relief holes has been temporarily successful. However, over time, cracks continue to grow and a more permanent repair is needed. As a part the bridge rehabilitation alternative this issue should be fully addressed. Each hole should be examined for any new cracks that have propagated through them using dye penetrant.
New relief holes should be drilled to blunt the tip of any new cracks found. The cracked areas should then be cleaned and painted. After painting, these areas should be retrofitted with structural steel stiffeners bolted to the girders to keep the flanges from rotating under uplift loads (See Figure 4-4 for recommended repair).

4.1.2.8 Tracks and Treads

The existing track and tread plate assemblies are in poor condition. They are heavily worn and corroded. Many of the bolts connecting the two plies of the curved tread plate assemblies have failed. The upper plate of the two-ply assembly is also designated as the bascule girder bottom flange. Many of the bolts fastening the tread plates to the bascule girder flange connection angles have also failed. Bolt failure has been an ongoing problem that began shortly after the bascule superstructure was reconstructed in 1987. The bolt failures are likely due to a combination of elastic and plastic deformations of the tread plates and relative movement between the two plies of plates during
cyclical loading when the bridge rolls open and closed. An additional contributing factor may be the plastic deformation of the tread and track plate contact surfaces. MDOT has welded side plates between the two plies of tread plates on their sides over their full length of roll to hold them together (See Photo S-22).

The flat track plate assemblies consist of an upper plate upon which the tread assemblies roll, and a lower masonry plate. Most of the bolts connecting those two plates have failed. Track plate movement is visible when the bridge is opened and closed. This movement is most notable in the northwest quadrant. Light gauge angles have been welded to the sides of the two track plates to hold them together over the full length of roll (See Photo S-23). The tread plates have indentations reflecting the countersunk holes in the track. This is an indication that the contact stresses exceed the capacity of the material.

The pintles in the track plates show significant wear. This wear is primarily due to longitudinal movement of the track plates that causes the pintles to become misaligned with the mating receiving holes in the tread plates. During the June 2013 site visit and inspection, loud grinding noises were observed to emanate from the mating surfaces of the track pintles and tread plate receiving holes during bridge opening and closing in the southwest quadrant.

For the bridge rehabilitation alternative, complete replacement of the two-ply track and track plates assemblies is recommended. To facilitate replacement of the tread plate assemblies, the associated bascule girder bottom flange angles to which they are connected should also be replaced.

When the bridge was originally constructed, steel grillage beams embedded within the bascule piers were provided to support the track assemblies. Though these beams are encased in concrete and not visible, they are potentially also in poor condition. During the 1987 bascule superstructure replacement when the current track plates assemblies were installed, it was reported that the top flange of at least one of the grillage beams was removed and replaced due to its deteriorated condition. For the bridge rehabilitation alternative, replacement of the grillage beams is also recommended. This work would require removal of the pier concrete that encases them.

### 4.1.2.8.1 Data Review

URS reviewed GeoMoS survey monitoring data for the track plate assembly of the bascule span’s northwest quadrant. That data indicates movement of that track plate assembly during bridge opening and closing. The maximum recorded lateral movement of the track plates is 0.36 inches. The two plies of track plates should be in intimate contact at all times and experience very little movement during bridge operation. This recorded movement is excessive and is a likely contributor to failure of the connecting bolts and the cracking of the reinforcing side plates. The maximum recorded vertical movement of the upper plate of the two-ply track plate assembly is 0.30 inches and could be due to a number of reasons. There are inadequate connectors still intact between the top and bottom track plates. The underlying pier concrete,
supporting track girder, and/or grout pad between the top of the track girder and bottom plate may be deteriorated.

Previous reports concluded that the tread/track assemblies do not have sufficient capacity to carry the weight of the bascule leaves when they open and close. A major shortcoming with the configuration of both the treads and tracks are the two individual plates that are bolted together instead of a single element with adequate thickness.

The heads of the bolts connecting the upper 3-inch tread plate to the side angles of the girder bottom flange are covered by the lower 3-inch tread plate and cannot be directly inspected or replaced. Figures 4-7 and 4-8 show this existing arrangement. The upper and lower plates have inadequate thickness and move with respect to one another. This movement caused a shear force across the mating surfaces of the two plates, which creates bending in the bolt shanks.

The underlying grillage beams that are embedded in the pier concrete of the originally constructed bridge do not provide adequate support for the track assemblies of the replacement superstructure that was constructed over them in 1987. This may be due to improper construction of the replacement bascule superstructure over the grillage assemblies, corrosion, failing grout, and/or providing stepped shims. Each of the individual track plates have inadequate thickness and exhibit deflection when the bridge opens and closes. This creates tensile stresses in the bolts of the track plate assembly. These bolts were also improperly detailed with the threaded portions of their shanks within the shear planes where they have to resist horizontal force. This causes excessive bending and shear stresses in these bolts.

A previous report in 1994 by Stafford Bandlow Engineering describes the tread assembly connecting bolt failures as follows: “The curved tread mounting bolts that have defects in the interface area where the two 3-inch plates come together were damaged in shank area. The two 3 inch curved plates were experiencing relative tangential movement between the two surfaces creating excessive shear in the mounting bolts and causing loud snapping noise as the leaf rolled back and forth during operation”.

### 4.1.2.8.2 Analysis of Existing Tracks & Treads

An evaluation of the existing tread and track assemblies indicates the thickness and material properties of each of the two plies of plates are inadequate for the heavy rolling load they need to carry when the bridge opens and closes. The 3-inch thickness of each track and tread plate is below the calculated 4.1-inch minimum thickness required based on the current AASHTO Movable Bridge Design Specifications. These plates have a yield strength of 50 ksi. Based on the AASHTO specifications, that strength is inadequate for the line bearing width provided by the plates. The shear stress in the pintles that extend from the top track plate exceeds the allowable stresses for that material. The heavy wear, indentations and plastic flow in the track and tread plates are the result of high bearing stresses that exceed the allowable stresses of the plate material.
4.1.2.8.3 Detailed Repair Recommendations for Tracks & Treads

As part of a bridge rehabilitation, the track and tread assemblies would be replaced. To enable that work, the bridge would need to be closed to traffic and the bascule leaves raised and placed on temporary shoring. This could be most effectively performed with the bascule leaves in the closed position during the non-navigation winter season. Jacking and shoring of each leaf could be performed at two locations. A rear jacking frame could be placed in the bascule pier pit and positioned under the back counterweight truss. A forward jacking frame could be provided under the bascule girder or first floorbeam in front of the bascule pier with it supported by temporary steel piles driven in the waterway. As discussed in Section 4.1.2.15, as part of a bridge rehabilitation, the existing timber dolphins in front of the pier would be removed and replaced with a new fender system. The temporary piles for the forward jacking frame could be driven after the existing dolphins were removed. Alternately, if the dolphins were still in place when the forward jacking frame needed to be installed, temporary piles could be driven on each side of them and a cap beam spanning between them provided to support a jacking frame. Figure 4-6 provides a schematic of this jacking and shoring arrangement.

Replacing the tread assemblies would require removal of the two 3-inch plies of tread plates, the bascule girder bottom flange angles and the girder web side plates. Figure 4-7 shows these components that would need to be removed. The roll radius of the bascule leaves and the elevation of the rolling surface at the bottom of the tread assemblies would have to be maintained. To meet this requirement with thicker replacement tread assemblies, the bottom of the bascule girder web plate would also need to be cut back. Portions of the bascule girder web stiffeners would also be removed to a point just above of the removed web side plates.

Preliminary calculations indicate that replacement of the two-ply track and tread plate assemblies with single piece 6-inch thick steel forgings having a minimum yield strength between 70 to 90 ksi would be appropriate for this bascule structure. Thicker single-piece tread and track...
elements perform better than multiple plies of thinner ones for rolling lift bascule bridges with under-deck counterweights that have relatively small roll radii. The curved tread plates and the flat track plates would both be forgings. The use of forgings is an effective and economical way to attain the required high yield strength for these thick components. Forgings would be preferable over castings because they avoid the concerns about potential internal defects that can occur within castings.

Six-inch single piece tread and track plates would satisfy the calculated 4.1-inch minimum thickness that would be required based on the AASHTO Movable Bridge Design Specifications. The somewhat greater than minimum required thickness would provide a degree of conservatism that is recommended based on historic behavior of track and tread assemblies on rolling lift bridges. Based on using 70 ksi yield strength for these components, the 12-inch effective line bearing that would be provided would be conservatively greater than the 6.1 inch minimum that would be required based on the AASHTO Movable Bridge Design Specifications.

The bridge’s existing track plates have a single line of separate circular pintles pressed into them that mesh with circular holes bored in the tread plates. Replacement track forgings would have integral rectangular lugs machined into them that would alternate from side to side. Mating receiving pockets would be machined into the replacement tread forgings. The integral lugs and mating receiving pockets would promote precise mating between the treads and tracks each time the bridge opened and closed. This would ensure accurate tracking of the bascule leaves for better position control compared to the current system with a single line of pintles in the track plates. The bearing and shear stresses in the integral lugs of the track plates extending above their rolling surface would also be conservatively below the allowable stresses for those forgings.

As noted above, the existing two bascule girder bottom flange angles would also need to be removed. They would be replaced with two new split flange-web weldments that would provide the means for attaching the replacement tread forgings to the bascule girder web by bolting. No field welding of these elements would be required. The flange-web weldments would be composed of vertical plates, curved flange plates, and radial stiffeners. Each split flange-web weldment would consist of a 3-inch thick flange plate full penetration welded in the fabrication shop to a 1-1/8 inch web side plate. The combined thickness of the vertical legs of the two flange-web weldments and the existing bascule girder web plate sandwiched between them of 3.38 inches would conservatively meet the requirements of the AASHTO Movable Bridge Design Specifications.

Rolled angles could not be used instead of flange-web weldments because of the need to provide thicker and more robust components compared to what currently exists. Also the required height of the vertical leg of these weldments is greater than what is available as a rolled angle section.
The existing and proposed replacement track and tread configurations are shown in Figure 4-8.

![Figure 4-8: Existing versus Proposed Track and Tread](image_url)

New stiffeners would be shop welded to the vertical and horizontal legs of the split flange-web weldments at the same orientation as the remaining portions of the existing girder web stiffeners such that they could be spliced together with bolted connections. After all-shop-welding, the flange-web weldments would be stress relieved prior to final machining.

Each pair of split flange-web weldments would be assembled and temporarily doweled together prior to final machining. The curved bottom of the pair of split flange-web weldments would be machined to match the machined radius of the mating top of the new tread forging. The dowels between the pair of split flange-web weldments would be utilized in the field to assure accurate re-assembly when attached to the bascule girder.

The tread forging will be temporarily assembled to the pair of split flange-web weldments in the shop while its final rolling radius is machined.

The split flange-web weldments would be bolted to the girder web plates in the field using the existing holes in the girder webs. No field welding to existing elements of the bascule girder would be required with this arrangement. The tread plate will be final-attached in the field to the split flange-web weldments with bolts.

Both ends of the split flange-web weldments would also be connected to the existing bascule girder bottom flanges. At the back end of the weldments, the connection would be made by bolting through the existing girder flange and new backing connection weldments attached to
each side of the existing girder web and stiffener. These weldments would be connected to the girder flange as well as the girder stiffener with bolts in double shear. The front end of the split flange-web weldments would be connected to the existing girder flange with a conventional bolted flange splice. This proposed arrangement for attachment of the replacement tread assemblies is shown in Figure 4-9.

Removing of the track assemblies would include removing of the two-ply track plates, sole plates and associated connecting hardware. The concrete embedding the track plate anchorage system and track girder would also be removed as well as the track girders. New track girders and track forgings would be fabricated, installed and precisely aligned.

Prior to field delivery, a numerical roll through procedure would be performed in the machine shop on the track and tread forgings to confirm precise alignment and mating between them. The alignment and mating would then be physically verified in the field by roll-through testing. Undersized bolts could be used to initially attach the track forgings to the split-flange and web weldments. This would allow for adjustments to be made if determined necessary by the roll through test. After final verification of tread and track alignment, concrete encasing the new track girders would then be placed.

4.1.2.8.4 Finite Element Analysis of Proposed Tread Assembly

A detailed non-linear finite element analysis was performed to evaluate performance of the proposed tread retrofit assembly, including the 6-inch thick tread forging, flange and web weldments and bolts connecting the tread forgings to the flanges. That analysis confirms that the proposed concept with a 6 inch thick tread forging is adequate to sufficiently distribute the loads and limit deformations in the assembly such that the stress range in the connecting bolts is well below their fatigue limit. Furthermore, the distribution of loads, results in compressive and contact stresses that are within the design parameters. See Appendix F for further description of the analysis.
4.1.2.9 Rack Frames

The rack frames of both leaves consist of a welded steel plate girder that ties into front and rear support columns. The lower portion of each rack support column, as well as the bottom of the diagonal struts, is embedded in the bascule pier concrete. The portions above concrete were replaced during the 1987 rehabilitation and are spliced into the original components just below the concrete surface, at track elevation. The rack gear castings are bolted to the bottom flange of the rack girders.

Since jacking of the leaves would be required to change out the treads, temporary removal of one of the three rack segments from each of the rack frames could facilitate this by yielding 4” of clearance for jacking. If more clearance than this is required, the pinions could also be removed to yield an additional 5.5”.

The rack frames are in good condition with no notable deficiencies.

4.1.2.10 Center Locks

A Scherzer-type center lock device transfers live loads between each pair of mated bascule girders as traffic crosses the center break between the bascule leaves. This is a non-mechanical “Jaw and Diaphragm” interlocking system.

The center lock assemblies are in good condition. However, the north center lock has a gap above the casting and the south center lock has a gap below the casting (See Photo S-19). The gap at the castings is approximately 1/2”. This is likely due to general wear and possibly improper shimming. There is likely wear of the jaw plates and castings that was not visible at the time of the follow-up inspection. The excessive gaps between castings and jaw plates produce “pounding” and impact at the joint as heavy trucks pass over the center break. Both center locks are adequately lubricated.

As a part of the rehabilitation alternative, in order to correct for wear and provide a tighter fit, the jaw wear plates should be removed, ground smooth to remove grooving, and then properly shimmed to better fit the castings. Shimming should also be performed at the four rear girder uplift points in coordination with the center locks in order to ensure that all four tails are hitting at the same time that the center lock castings and wear plates are at a proper orientation. This orientation should provide approximately 1/16” of clearance from the jaw wear plates to the castings.

4.1.2.11 Counterweights

The counterweights of the bascule spans are connected to the rear of the bascule girders by vertical and horizontal trusses. The counterweight itself is a reinforced concrete mass with three upper and three lower pockets for housing lead balancing adjustment blocks.

The concrete counterweight and supporting truss system were in good condition without notable deficiencies. A bridge rehabilitation would require re-balancing work including modifications to
the lead balancing block arrangement. No other structural work would be required for the counterweights.

4.1.2.12 Bascule Piers

As part of the rehabilitation in 1987, epoxy crack injection and formed concrete repairs were performed on both bascule piers’ exterior walls and interior counterweight pit walls. The upper parapet wall portions of both piers were removed and replaced to accommodate the superstructure replacement and the new operator house at the west bascule pier. During the 2006 rehabilitation, additional patch forming and crack repairs were made on the exterior of the bascule piers. In general, the crack and formed concrete repairs previously made are in good condition. However, several locations of map cracking, spalling and patch delamination are present.

The upper cap nosing on the south side of both bascule piers exhibits delamination and heavy scaling for a height of two feet over the full width of the pier. The west elevation of the west bascule pier has a patch at the south end that is exhibiting minor map cracking.

The east bascule pier has several spalls, locations of delamination and map cracking. The west face of the east bascule pier has a two-foot diameter by two-inch deep corner spall at a patch located on the south end of the pier (See Photo S-31). The same face of the pier has a six-inch high, by six-inch wide, by two-inch deep corner spall at the upper cap nosing on the south end. A large patch delamination and map cracking is evident on the east bascule pier’s east elevation.

Many of the previous crack repairs located in the interior of both bascule pier counterweight pits are in good condition (See Photo S-30). However many are leaching and have efflorescence. As part of the rehabilitation alternative, the areas of failed formed concrete patches would be removed down to sound concrete, the existing reinforcing exposed, and new formed concrete applied. Cracked concrete would be flushed and epoxy injected.

A side effect of the problematic movement of the track plates upon which tread segments roll is that a small lateral misalignment of the bridge leaves has created a clearance issue at the north wall of the west pier pit and the south wall of the east pier pit when the bridge opens. The edge of the bridge deck at the longitudinal break rubs on the wall just below the top of the concrete mass pour at track level. Review of the original pier plans and the 1987 superstructure plans revealed that the theoretical clearance is only 1/2” between these two surfaces. Small initial design clearances combined with changes to the structures alignment has resulted in the rubbing interference. Hand chipping of the concrete has been performed to alleviate the rubbing (See Photos S-27 & S-28). During the walk-through inspection, it was observed that clearances in these areas are still not sufficient and appear to be touching in some locations. (See Photo S-29). For the bridge rehabilitation alternative these areas should be chipped back further and a formed concrete repair made that would provide at least 1 inch of clearance.
The bascule piers are supported on closely spaced timber piles. 296 piles support the west bascule pier and 312 piles support the east bascule pier. One five-foot thick concrete seal, one four-foot thick concrete footing and one 5'-9” thick concrete pit floor form the pile cap and base of the piers. The 1987 superstructure replacement resulted in increased dead load of approximately 350 kips per bascule girder at both piers. The pile foundation of the west bascule pier was analyzed to determine the pile loads for two loading conditions. First, when the bridge is closed with vehicle loading on the bascule and approach span. Second, when the bridge is open with vehicle loading on the approach span, and full longitudinal wind loading applied to the open bascule span. It was determined that the governing pile loading is realized when the bridge is closed. The maximum pile load is approximately 25 tons when considering Load Factor Design (LFD) service load combinations. The existing pile capacity is unknown since the plans do not indicate it and no driving records are available. However, it is quite typical for these types of piles to have a capacity of more than 30 tons. Considering this and the fact that no settlement of the pier substructures have been noted, it appears that the pile foundation is adequate for continued service even with the heavier superstructure.

Temporary shoring loads on the counterweight pit floor need consideration for the bridge rehabilitation alternative. In order to replace the bridge tracks and treads as discussed in a previous section, each leaf of the bascule span superstructure would be left in the closed position and jacked up onto a rear shoring tower and forward temporary piling outside of the pier. The rear shoring tower would carry the load directly down to the counterweight pit floor. Review of the original plans shows that the thickness of the mass pour concrete from top of pit floor elevation to the top of timber piles is 9 feet 3 inches. This thickness is more than adequate to safely distribute the shoring loads to the piles without concern for failure of the floor concrete. Additionally the shoring loads would be required to be spread out on the concrete surface using timber cribbing to reduce the local pressure on the concrete.

Concrete cores of 3 3/4” diameter were taken from the bascule pier pits to determine concrete strength, composition and degree of deterioration of these old foundations. At the east bascule pier, the cores were taken from the pit floor and both mass pour walls below the embedded track girders. At the west bascule pier the cores were taken from the west wall as well as the north and south mass pour walls below the track girders. A petrographic examination was performed and determined all the cores exhibit similar composition of natural gravel coarse aggregate and natural sand fine aggregate with portland cement paste. The test determined the concrete is not air-entrained, but no signs of freeze-thaw damage were evident. A minimal amount of alkali-silica reaction (ASR) has occurred in the samples from the east bascule pier pit floor and west bascule pier west wall, but only a minor amount of ASR-associated deterioration has occurred. The concrete samples not used in the petrographic analysis were tested for compressive strength. The east bascule pier pit floor samples strength ranged from 7,310 psi to 9,280 psi. The east bascule pier wall samples strength ranged from 4,940 to 10,960 psi. The west bascule pier wall samples strength ranged from 5,830 psi to 8,560 psi.
See Appendix G for the full petrographic report.

4.1.2.13 Slope Protection

The abutment areas have a slope protection system at each of the four bridge quadrants consisting of slope paving and heavy riprap. The 1987 rehabilitation plans call for heavy riprap to be applied from elevation 586 to the channel bottom at each quadrant of the structure. There is currently only a small covering of riprap close to the abutments, which does not appear to match the quantity called for in the 1987 rehabilitation plans. The slope paving is no longer fully effective, with notable deficiencies at the south sides of both abutments. The paving is becoming undermined at the west abutment and deterioration with exposed rebar is visible at the east abutment (See Photos S-40 through S-43).

It is recommended that as a part of a rehabilitation alternative, the existing slope paving be removed at all quadrants of the bridge and replaced with a covering of heavy riprap. The heavy riprap would help to stabilize the channel bank and prevent any future erosion that could lead to problems at the abutments.

4.1.2.14 Scour Countermeasures

The structure is classified as scour critical and is currently scheduled for underwater inspections at regular 60-month intervals. Prior to the 1987 rehabilitation, the upstream sides of the bascule piers were found to be undermined with the supporting timber piling exposed. The 1987 rehabilitation included providing scour countermeasures to address that finding. Grout backfill was placed beneath the undermined portions of the foundation seals of the bascule piers. Grout filled bags with a stone covering was provided along the portions of the faces of the bascule piers adjacent to the undermined regions. The rehabilitation plans indicate that the stone covered grout filled bags were to extend 2’-6” above the bottom of the seals.

Another underwater inspection was performed in 2010. It provided updated information about the underwater portions of the substructures and the condition of the surrounding waterway bottom. That inspection report’s substructure drawings showed that general aggradation and degradation had occurred to the bottom of the waterway, but the scour countermeasures installed in the 1987 rehabilitation were still effective and no new undermining of the bascule piers had developed.

A hydrographic survey was performed at the bridge in 2013 after high water conditions occurred during the spring of that year on the Saginaw River. The survey revealed two new large scour holes have formed on the downstream side of the bascule span. One scour hole was estimated to be 20 feet deep, 50 feet long and 30 feet wide. The center of that hole is approximately 45 feet northwest of the east bascule pier’s northwest corner. The second scour hole was estimated to be 15 feet deep, 25 feet long and 25 feet wide. The center of that hole is approximately 44 feet east of the west pier’s northeast corner. The survey also revealed that the upper portions of the footings at both bascule piers were exposed more than what was observed in the 2010
underwater inspection though no undermining was identified. As part of a bridge rehabilitation, scour countermeasures should be performed to address the downstream pier ends near where the two current scour holes are located.

Based on input by MDOT’s Hydraulics Unit, a recommended scour mitigation method to address these holes would be to provide riprap mats on geotextile fabric. Initial sizing for riprap was performed in 1995 and determined that heavy riprap (D50= 16”) would be needed. The riprap mats would extend 25 feet from the downstream and navigation-channel faces of each bascule pier where the scour holes have formed. The riprap would be placed in three layers with a 4-foot-by-4-foot header below the pad along its outer perimeter. The portions of the scour holes beneath the proposed riprap pads would also need to be filled.

With the water depth in this area being approximately 24 feet, installing geotextile fabric and riprap headers would be difficult. Adding to the challenge of implementing this scour mitigation is a submarine cable that was installed under the navigation channel between the downstream ends of the bascule piers as part of the 1987 rehabilitation. Great caution and protective measures would need to be taken when performing the scour mitigation work adjacent to and over the cable to minimize the risk of damaging it. Alternately, the submarine cable could be replaced, however the cost for doing so would likely exceed $500,000.

This bridge site has a significant scour history that has been demonstrated by the scour and undermining on the upstream ends of the bascule piers that was addressed during the 1987 rehabilitation and the two new large holes that formed during 2013. Additionally, the calculated scour for the 100-year event is over 40 feet. Based on these considerations, the proposed countermeasures to address the two recently formed scour holes would likely not be considered sufficient to remove the scour critical classification of the bridge.

4.1.2.15 Fender System (Pier Protection)

Each bascule pier is protected by a set of five individual dolphins; three 6’-0” diameter, sand filled, steel sheet pile dolphins with a 3’-0” thick concrete cap, and two pile cluster dolphins consisting of 12” to 14” diameter timber piles with a total of 37 in each cluster (See Photos S-38 & S-39). The steel sheet pile dolphins were part of the original construction, while the timber dolphins were installed in the 1987 rehabilitation project. URS observed the physical condition of the dolphins from a boat and found them to be satisfactory but with some spots of dry rot and minor damage.

The dolphin fender system was evaluated in an earlier study and the results included in the 1999 In-depth Structural Inspection Report. The analysis followed the guidelines set by AASHTO Guide Specification and Commentary for Vessel Collision Design of Highway Bridges. This evaluation considered the typical type of vessel commonly found in the Saginaw River, determined the design vessel impact energy and determined the energy dissipation capacity of both types of dolphins.
Protection cell dissipation energy is a combination of section strength, acting as a cantilever, and allowable impact deflection. The criterion is that the deflected position after impact should not be in contact with the bascule pier, which could potentially damage the entire structure. The existing clearance from the bascule pier exterior concrete face and the interior edge of dolphin is limited; 2’-9” separate the center interior sheet pile dolphins from the bascule piers, a range of 11 inches to 3’-11” at the two timber pile cluster dolphins, and the exterior sheet pile cells have a clearance ranging from 3’-6” to 6’-9”.

The design vessel selected was the “Adam E. Cornelius”, a 28,200 metric ton dead weight tonnage (DWT) vessel with a length of 680 feet and a 78 foot beam width. The design impact speed was found to be 4.5 miles per hour, with a vessel impact energy for a head-on collision of 61,070 kip-ft. Only the outermost sheet pile cell dolphins are susceptible to head-on collisions. The interior three dolphins have a maximum possible angle of impact of 6°, which results in an impact energy of 336 kip-ft. URS reviewed the bridge operator’s reports from 2011 and 2012 and confirmed that the design vessel used in the previous analysis is appropriate. Lake freighters such as the “Lewis J. Kuber” now traveling the Saginaw River have a DWT capacity of 22,300 metric tons.

The center sheet pile dolphin cell was found to have an energy dissipation capacity of 22 kip-ft., when considering the cell section properties and clearance available for deflection. The timber pile cluster dolphin’s individual capacity was found to be 102 kip-ft. when considering the maximum clearance for deflection. These calculated capacities of individual dolphins fall far short of the design vessel impact energy from both head-on collisions and indirect angle collisions. As was recommended at that time, a new fender system should be considered for inclusion in a major rehabilitation project.

The proposed pier protection system is similar to what is used on other Great Lakes waterways and consists of eight 25’-0” diameter sheet pile cells, two at each bascule pier approach with a steel pile supported fender wall spanning between each cell and across the front face of the bascule pier, while maintaining the existing 150’-0” horizontal clearance. A walkway would be installed along the top of fender wall from cell to cell for accessing navigation lights and general maintenance. This system has a layout to protect the entire north and south faces of bascule piers from head-on collisions, and would guide the ship if an impact is at an indirect angle. See the proposed bridge plan in Exhibit D-2 in Appendix D for the layout.

### 4.1.3 Operator’s House

The operator house is situated on the north side of the east bascule pier. The house has four floor levels. From the top down these are the operator’s room, the control room, the equipment room, and the generator room. The operator house was replaced as part of the 1987 rehabilitation and no significant deficiencies are noted.
For the bridge rehabilitation alternative however, it is recommended that the roof, doors and windows be replaced since they will be roughly 30 years old at that time, an age where replacement is normally needed.

### 4.1.4 Approach Pavements

Approach pavements to the east and west of the bridge consist of approximately 45-feet of a 10-inch reinforced concrete pavement. Beyond the concrete pavement section lies bituminous approaches. Sidewalks continue beyond the bridge and are maintained throughout each approach. The concrete approaches are in relatively good condition with various locations of concrete sidewalk settlement. Both bituminous pavement approaches are also experiencing settlement where the asphalt pavement section stops and the concrete approach begins (See Photos S-9 and S-15). Minor repairs to fix settlement and transition to any repairs would be included as a part of a structure rehabilitation.

### 4.2 MECHANICAL SYSTEM

The following subsections provide recommendations for rehabilitation of the mechanical system of the movable span.

#### 4.2.1 General Description of Mechanical System

The movable span of the bridge consists of two rolling lift bascule leaves. Curved tread plates are attached to the bottom flange of the heel portions of each of the two bascule girders of each leaf. These tread plates roll on flat horizontal track plates that are mounted into the concrete of the bascule piers. The curved tread plates have circular pockets machined into them. The horizontal track plates have pintles pressed into them that mesh with those pockets to prevent slippage as the leaf rolls back and forth. The center of radius for the tread plates is the center of rotation and roll for each bascule leaf. Output drive shafts of the machinery system pass through the web of each bascule girder. These shafts are situated at the bascule leaf’s center of rotation and have a pinion attached to their ends. As the pinions rotate, their teeth engage and drive against those of the stationary racks to cause the bascule leaf to roll open and closed.

The mechanical system for operating each bascule leaf is powered by two 30 horsepower electric motors. The torque from the output shafts of these motors are transmitted into an enclosed primary differential speed reducer whose output shafts in-turn drive two enclosed secondary speed reducers located on each side of the primary speed reducer. The output shafts from each secondary speed reducer transmits torque to the drive pinions attached to their ends that in-turn engage the straight racks attached to pier mounted stationary frames situated on the outboard side of the bascule girders (see Photo M-1). Refer to Figure 2-2 for a schematic diagram of the operating machinery.

The bascule leaf braking is provided by one motor brake and one machinery brake. The two brakes for each leaf are identical and act on the primary reducer’s input shaft.
4.2.2 Rack and Pinion Systems

There are two sets of racks and pinions per bascule leaf. The main drive pinion, P1, is mounted on the end of Shaft S1 adjacent to the web of the bascule girder. The shaft is supported by Bearings B1 and B2. Bearing B1 is mounted to the bascule girder web at the center of rotation of the leaf. Bearing B2 is supported on the machinery floor structure. The racks are attached to the rack girder located on the pier. Three segments that make up each straight rack.

Observation of the pinion gears while the bascule leaves are being opened and closed along with drift testing each leaf indicates there is a limited amount of imbalance for each bascule leaf. The leaves are slightly tip heavy near the closed position and are slightly counterweight heavy toward the fully open position. The pinions drive the spans while opening and closing the leaf, moving from driving to retarding at different points along the rack, which could be attributed to the issues with the limited amount of imbalance as well interference in mating of the track pintles and tread receiving pockets. A slightly span-heavy condition is desirable when the span is in the closed position because it promotes the span remaining closed and staying seated against its live-load uplift supports. A slightly span-heavy condition exhibited by a bascule girder tip reaction of approximately 2,000 lbs. is desirable.

The bolts that attach the rack forgings to the supporting rack girders are secure and in good condition. There are a few nuts with minor surface corrosion. The operating rack and main pinions are well lubricated (See Photo M-2).

As part of a rehabilitation project, one of the rack segments would have to be temporarily removed in order to jack the leaf to replace the track and treads. After making the repairs to the track and tread, the rack would be bolted back in place using the existing turned bolts and shims to maintain the original alignment to the remaining two rack segments per each side. If, after the work is complete, the contact between the rack and pinion is not as desired, the rack would be adjusted to obtain the required contact percentage between the rack and pinion.

4.2.3 Speed Reducers

The primary enclosed differential speed reducer consists of a single input shaft extending through either side of the reducer being driven by a motor on either side. The motor and machinery brake drums are located on the primary reducer input shafts, between the motor coupling and speed reducer housing. The input shaft drives two sets of gears submerged in an oil bath with supplemental pressurized lubrication systems for the bearings and gears. The last gear set, which is the differential gear set, drives an output shaft that extends through either side of the reducer, driving each of the secondary enclosed speed reducers to either side. The differential gear set acts to distribute torque evenly to each secondary reducer throughout travel.
The secondary enclosed speed reducers consists of a single input shaft extending through one side of the reducer, driven by the primary reducer, with the output shaft extending through the opposite side of the reducer, driving the S1 Pinion Shaft. The input shaft drives two sets of gears submersed in an oil bath with supplemental pressurized lubrication systems for the bearings and gears.

All of the speed reducers appear to be in good operating condition. There is minimal wear on the gear sets and the lubrication systems are operating normally. Both of the primary reducers have minor oil seepage around the input and output shafts (See Photo M-3) while the secondary reducers have minor oil seepage around the input shaft only (See Photo M-4).

As a part of the rehabilitation, the input and output seals of the primary speed reducer along with the input seals of the secondary speed reducers would be replaced. In order to replace the primary reducer output shaft seals and the secondary reducer input shaft seals, the floating shaft couplings would need to be removed and replaced when completed.

4.2.4 Brakes

There is one motor brake and one machinery brake per leaf. Both of the motor and machinery brakes are identical General Electric (GE) electro-hydraulic thruster released, spring set brakes (See Photo M-5). GE no longer manufactures these brakes, nor are there spare parts available if there were to be a failure with the hydraulic thruster unit or the brake actuator mechanism. All of the brakes are in good condition and working properly at this time.

As part of the rehabilitation, due to obsolescence, the brakes would be replaced with new electro-hydraulically operated brakes. In order to replace the brakes the motors and motor couplings would have to be removed to in order to remove the existing brake wheels and install new brake wheels at the same location for the new brakes.

4.2.5 Shafts, Bearings and Couplings

4.2.5.1 Shafts

Each set of drive machinery consists of two pinion shafts, S1, and two floating shafts, S2. Overall, the shafts are in good condition. The pinion shafts have minor surface corrosion forming near the B1 bearing location (See Photo M-6).
As part of the rehabilitation, the pinion shafts would be cleaned and painted. Due to the proximity of the pinion bearing sand blasting should not be used. The shaft would be hand tool cleaned and painted with an epoxy mastic aluminum primer suited for the cleanliness achieved.

4.2.5.2 Bearings

Each set of drive machinery consists of two B1 bearings and two B2 bearings. The bearings are of the spherical roller type in enclosed housings and appear to be in good operating condition. The B1 bearing housings have minor surface corrosion forming on the housing (See Photo M-6). As part of the rehabilitation, the bearing housings would be cleaned and painted. Due to the proximity of the seals, sand blasting should not be used. The housing would be hand tool cleaned and painted with an epoxy mastic aluminum primer suited for the cleanliness achieved.

4.2.5.3 Couplings

Each set of drive machinery consists of two C1 couplings attaching the secondary reducer output shafts to the pinion shafts, four C2 and C3 couplings attaching the floating shafts to the output of the primary reducer and the input shafts of the secondary reducer and two C4 motor couplings attaching the motors to the primary reducer input shaft. The couplings are in good condition and appear to be well aligned. The C4 motor couplings are leaking grease from the end seals and gasket (See Photo M-7).

As part of the rehabilitation, the C4 motor couplings would be disassembled and the gaskets and seals replaced. Due to the C2 and C3 couplings being disassembled and removed from the shafts to replace the speed reducer seals, the seals and gaskets for these couplings would be replaced at that time.

4.2.6 Rear Locks

There are four rear locks, two for each leaf, located at the back of the leaf, below the counterweight. The rear lock consists of a strut rotating about a pinned connection to the base plate. On top of the strut is a rocker, which is also rotating on a pin. As the strut is driven into place, the rocker contacts the strike plate located on the bottom flange of the main girder and pivots about the pin. As the rocker pivots, the overall length of the rear lock increases, pushing the back of the leaf up. This causes the live load shoes to contact the uplift girder and preload the live load.
load shoe assemblies. This reduces the amount of impact seen by the live load shoes due to traffic on the leaf. All rear lock assemblies are operating properly and are in fair condition. The actuators are corroded, but are not affected functionally (See Photo M-8). DOT Maintenance Personnel indicated the motors on the rear locks had been replaced recently and the manual operators freed up. This is common for this type of actuator when manual operation is not performed frequently. There is heavy corrosion on some of the strike plates (See Photo M-9) and their fasteners (See Photo M-10). Some of the limit switch brackets are also corroded to the point they are about to fail (See Photo M-11).

As part of the rehabilitation, the rear lock actuators would be replaced with hydraulic cylinders and remote hydraulic power units (HPU’s). This would allow the HPU to be placed away from the sidewalk break where salt water drips through during the winter. The hydraulic cylinder would be easily replaced periodically due to corrosion. The lines and hardware from the HPU to the cylinder would be stainless steel and rubber hoses at each end to facilitate movement and vibration. Each of the four individual HPU’s would be equipped with a manual hand pump integrated into the HPU tank and piping system for emergency operation. The limit switch brackets and other small hardware would be replaced with stainless steel brackets and hardware. The rear lock strut and rocker would be blast cleaned and painted. The strike plates would be removed, cleaned and painted. Galvanized bolts and stainless steel shims would be used to re-attach the strike plates to the main girders and the strike plate shimmed to obtain a pre-load on the rear live load shoes.

4.2.7 Summary of Mechanical Condition

Overall, the mechanical system for each of the bascule leaves is in good condition. The machinery is capable of operating the leaves smoothly and without issues. There are a few leaking seals and gaskets, but this does not affect the overall function of the operating machinery. If not replaced, the speed reducer seals that are leaking will get worse and become an issue for maintenance to deal with. Although in good working condition, it should be noted that the brakes are obsolete and replacement parts are no longer available for these brakes, as a result, the brakes would be replaced.

4.3 ELECTRICAL SYSTEM

The bridge’s electrical system is in fair to good condition with the exception of the drive system. The east (near) backup drive is not functioning and east primary drive must be reset frequently. Much of the time when designing electrical rehabilitations, the equipment is not in poor condition, but has become obsolete, which leads to serviceability problems and a lack of replacement parts. This is usually the case with variable speed drives, operator interfaces and PLCs. Items such as relays, starters and pushbuttons tend to be interchangeable with replacement devices. Other items such as span and lock limits receive heavy exposure to the environment and should typically be replaced. See Appendix E for electrical Photos.
4.3.1 Traffic Control Devices

The traffic gates are in fair to good condition. Each gate has some corrosion and appear to have been modified with proximity limits for gate position rather than the typical cam limits. Traffic gates are a critical component to a bridge operation. For another 50 years of operation, new traffic gates are recommended along with new LED style traffic signal heads.

4.3.2 Navigational Devices

The bridge’s pier and center channel navigational lights are in good condition and are not in need of replacement. However, because of replacing the bridge’s fender system and the extent of other structural repairs on the bridge, the lights should be replaced rather than risking them not being damaged or lost during construction.

New LED pier lights and span-mounted channel lights are recommended to replace the existing incandescent light fixtures. New LED fixtures are cost effective, long-lasting, and energy efficient with a 10-year bulb life. In addition, new bridge communication devices would be installed that include a marine radio and air horn.

4.3.3 Span Drive System

With rehabilitation, the existing main motors would remain and new variable speed DC drives would be installed to replace the non-functioning backup drives and the existing drive with intermittent tripping problems. The new system would be set up to alternate the drives each time to help ensure that there is always a backup.

4.3.4 Control System

A new electrical control system would include a new programmable logic controller (PLC) cabinet and modifications to the existing control console and motor control center (MCC). The existing PLC system is an Allen Bradley SLC model, which has been used since the early to mid-1990’s. It is still readily available, but is slowly being marketed and replaced with new generation PLCs. Because of this, the SLC model would likely be obsolete before the next rehabilitation.

A programmable logic controller (PLC) is recommended as the primary system to control and interlock the operation of the bridge.
backup simple relay control system would also be included as a redundant means of operation in the event of a failure in the PLC system. The relay system would involve more operator steps in order to perform a bridge opening.

Modifications to the existing control console would likely be necessary to interface to the new control system. In addition, a touch screen human machine interface (HMI) device would be installed to provide alarm and diagnostic information for the operator and bridge maintenance staff.

Limit switches, sensors and transmitters are common points of failure with any bridge control system, thus they would be replaced during rehabilitation.

**4.3.5 Motor Control Center**

The existing motor control center is not in poor condition, but it will be close to 30 years old at the time of the next rehabilitation. Circuit breakers can degrade over time, so it is recommended to replace the MCC to ensure the electrical components are properly protected.

**4.3.6 Generator**

The existing generator is installed on the bridge in one of the lower levels of the operator house. The generator system is in good condition, but the room it is located in, is too small to meet current code requirements and generator shops will not guarantee repairs because of the limited size of the room. A new generator mounted in an outdoor low noise enclosure is recommended. The new generator would be installed on a generator pad near the east abutment.

**4.3.7 Miscellaneous**

The submarine cables were installed in the 1987 rehabilitation project and still appear to be in good condition and would not be replaced at this time. Droop cables are used for electrical conductors between the pier and machinery room of the span. These cables are flexible and move during an operation and should be replaced during an electrical rehabilitation.

Lighting and receptacles would be upgraded as needed based on condition and code related issues. Some of the conduit and conductors would be able to be re-used, but it is recommended to replace any conduit that is exposed to the elements.
4.4 REHABILITATION SUMMARY

The bridge rehabilitation alternative would rehabilitate the entire bridge including structural, electrical, and mechanical components as well as offer techniques for addressing scour at substructure units. Major items that affect the cost of rehabilitation would be repairing the track and treads and installing an appropriate vessel collision system. Other notable costs for rehabilitation are balancing the bascule leaves, replacing the bridge railing, electrical rehabilitation and performing a partial painting of structural steel. The majority of work would be performed on the bascule span, because the approach spans are in relatively good condition.
5 BRIDGE REPLACEMENT

5.1 BASIC REQUIREMENTS

A replacement bridge would have an overall arrangement similar to the existing structure with one approach span flanking each end of a bascule span. The vertical and horizontal alignment of the existing bridge conform to MDOT geometric standards and therefore could be maintained with a replacement structure. The potential to raise the roadway vertical profile somewhat and increase the movable span’s underclearance in the closed position to reduce bridge openings could be studied in greater detail during the preliminary engineering phase for a bridge replacement.

Based on coordination with the United States Coast Guard, the movable span provides a 150-foot wide navigation channel and a vertical underclearance in the closed position of 20 feet above the Low Water Datum (LWD) at the center of channel. The agency indicated these clearance would be considered the minimum required for a replacement bridge. The 1987 superstructure replacement design plans indicate that the clearance above LWD is only 19.4 feet. For the purposes of this study, the underclearance in the closed position for a replacement bridge is assumed to have meet that of the existing bridge.

5.1.1 Single-Leaf versus Double-Leaf Bascule Span

A double leaf rolling lift bascule structure would be the most effective movable span for a replacement of the Lafayette Avenue Bridge. Because this style of bascule structure concurrently rotates and rolls backward when opening, a shorter superstructure is required to accommodate the same width of navigation channel compared to a trunnion style bascule.

A double-leaf bascule span would be consistent with the current movable span. The main girders of a double-leaf bascule span as propped-cantilevers when the two bascule leaves are mated. The heel portion of each girder is supported by uplift girders when the bridge carries traffic. With this configuration, the largest stresses occur at the heel section of the bascule girders where they are the deepest. The lowest flexural stress occurs at the tip of each leaf and allows bascule girders to most shallow at the mid-span. This enables the waterway clearance for the bridge in the closed position to be maximized at the center of the navigation channel where it most needed.

A single-leaf bascule structure would have similar primary components as a double-leaf although some some of them would be larger. To provide a minimum clear navigation channel of 150 feet, a single-leaf structure would require a span length in excess of 175 feet. This span length for a single-leaf bascule structure would require that the bascule girders be much deeper than those of a double-leaf structure. That greater depth would reduce the amount of vertical underclearance that could be provided in the closed position without raising the vertical profile of the structure and of its approach roadways and nearby intersections.
5.2 Recommended Replacement Bridge

5.2.1 Structure Configuration Layout

Exhibit D-2 in Appendix D provides plan and elevation views of a proposed structure for the bridge replacement alternative. Single span approaches comprised of precast prestressed concrete girders supporting a concrete deck would flank each end of a double leaf bascule span. The bascule span framing system would consist of floorbeams and stringers supported by two variable depth welded bascule plate girders. A half-filled steel grid deck with a 2-inch concrete over-fill would provide an effective deck system for the front arms of the bascule leaves.

The new structure would be built on the same vertical and horizontal alignment as the existing bridge with the existing vertical and horizontal navigation channel clearances maintained. The proposed replacement bridge would have an out-to-out width of 53’-3”. The clear roadway width of 40’-0” would accommodate three 12’-0” traffic lanes plus a 2’-0” distance from the edge of outside lanes to the face of the curb. Outside of the roadway, 5’-0” wide raised sidewalks accommodate pedestrians. Exhibit D-3 provides typical cross section views through the approach spans and the bascule span for the bridge replacement alternative.

5.2.2 Approach Spans

As a part of a bridge replacement alternative, the approach spans would incorporate precast prestressed concrete beams due to their cost effectiveness and low maintenance. The approach spans would have a reinforced concrete deck with 2% cross slope. The approach spans would have five-foot wide concrete sidewalks on either side of the roadway and a new four-tube bicycle bridge railing per MDOT Standard Plan B-26.

Integral abutments founded on steel piles driven to bedrock would support the ends of both approach spans.

5.2.3 Bascule Span Deck

The proposed replacement alternative would have a half-filled steel grid deck with a two-inch over-fill on the front arm portions of each bascule leaf. The over-fill concrete with this configuration acts as a wearing surface, protects the underlying steel grid, and can be scarified and replaced in the future. The steel grid would be bolted to the supporting stringers to promote ease of installation and enable removal and replacement if ever required in the future.

The heel portion of each leaf of the bascule span would have reinforced concrete decks from the rear breaks to near the front face of the bascule substructures. The concrete deck over the heel would provide additional weight to help counterbalance and be a protective roof over the machinery room and counterweight pits situated below it.
5.2.4 Bascule Span Sidewalks

The proposed bascule span would utilize a slip resistant galvanized steel plate sidewalk system. These sidewalk systems provide good traction and are relatively lightweight. A bicycle railing system would be installed along the sidewalk fascia within the limits of the bascule span. The railing would be equal in height to the four-tube railing on the approach spans, and have an additional tube provided to ensure that the 6” sphere pass through test is satisfied.

5.2.5 Bascule Span Structural Framing System

The structural framing system for a double-leaf deck-girder bascule span would consist of two bascule girders for each bascule leaf that carry all the movable span self-weight and traffic loads. The proposed framing system in the front-end of the bascule span would consist of longitudinal stringers that transfer loads into transverse floorbeams that frame into the bascule girders. The structural system outboard of the bascule girders would consist of sidewalk stringers and cantilevered brackets that transfer sidewalk loads into the bascule girders. The heel of the proposed structural system would consist of a concrete counterweight and counterweight trusses that connect it into the tail sections of the bascule girders. Forward of the counterweights would be the machinery floor with its steel framing spanning between the front counterweight truss and a floorbeam truss near the channel side pier wall. Rack frames would be outboard of the bascule girders and mounted rigidly to the bascule piers similar to the existing bridge. Live load uplift supports for the girder tail ends would be heavy steel brackets connected to the rear rack columns and braced to the pier walls.

The proposed tracks and treads would be made of high strength forged steel. The curved tread forgings would be bolted directly to the welded curved bascule girder flanges using shop installed turned bolts. The tracks would be bolted to steel grillages embedded in the pier concrete. The mating rolling surfaces of the track and tread components would have machined integral lugs in the tracks and mating receiving pockets in the treads along both the inner and outer edges of the elements.

Center locks that connect the tips of the east bascule girders to the west girders would be jaw and diaphragm type similar to the existing bridge, with the castings on the west leaf that lock into the east leaf jaws. This style of center lock has no machinery and there requires less maintenance and troubleshooting.

5.2.6 Bascule Piers

The replacement alternative would have fully enclosed counterweight pits similar to the existing bridge. The counterweight pits would be larger than currently exists to accommodate the larger counterweights. As a result, the overall size of the bascule piers would be greater than the size of the existing piers. The footings of the piers would be set so that the top of footing is located at the elevation of the existing channel bottom. Below the footing, it is anticipated that a concrete seal of approximately 12 feet would be required for cofferdam dewatering. Therefore, the bottom
of seal is located at approximately elevation 534.1 ft. The Scour Critical Bridge Action Plan states that the calculated Q100/Q500 pier scour depth is at approximately elevation 550. This is well above the estimated bottom of seal, which is desirable since the bridge would no longer be considered scour critical. The piers would be supported by steel piles driven to refusal in bedrock.

New bascule piers would be approximately 4 feet wider than existing and as a result, constrict the waterway opening. Construction in a floodway permitting sets limits as to the amount of allowable backwater. For the purposes of this study the east and west abutments are located 4 feet behind the existing as an estimate of what would be required to offset the wider piers and provide the same waterway opening.

5.2.7 Fender System

The channel is skewed approximately 7.5 degrees. The proposed pier protection system would be similar to what is used on other Great Lakes waterways and consists of eight 25’-0” diameter sheet pile protection cells, two at each bascule pier approach with a steel pile supported fender wall spanning between each cell and across the front face of the bascule pier, while maintaining the existing 150’-0” horizontal clearance. A walkway would be installed along the top of fender wall from cell to cell for accessing navigation lights and general maintenance. This system has a layout to protect the entire north and south faces of bascule piers from head-on collisions, and would guide the ship if an impact were at an indirect angle. See the proposed bridge plan in Exhibit D-2 in Appendix D for the layout.

5.2.8 Operator House

The new operator house would be similar in size to the existing but situated on the south side of the east bascule pier. Since the operator house requires a wider pier, locating the house on the south side provides the ability to accommodate the skew of the new fender system without significantly increasing the span length of the bascule bridge.

5.2.9 Approach Pavements

Bridge approach pavements would be reconstructed as a part of a bridge replacement. The new approach pavement would be constructed per Standard Plan R-45 and would match the new width of the bridge section.

5.3 MECHANICAL

The replacement machinery system would be similar in configuration to the existing system in place now with a separate drive system provided in a machinery room on each bascule leaf.
5.3.1 Machinery Layout

Figure 5-1 provides a schematic for the proposed machinery layout for the bridge replacement alternative.

![Figure 5-1: Mechanical Machinery Layout for Replacement Bridge](image)

5.3.2 Motors, Speed Reducers, and Racks & Pinions

There would be two 50 horsepower motors driving a primary enclosed differential speed reducer. The primary reducer would then drive two secondary enclosed speed reducers located either side of the primary reducer. Each of the secondary reducers would then drive the main pinion shafts, which would move the leaves by torque transfer through the rack and pinions. All the machinery would be sized per AASHTO for the leaf configuration mentioned herein. The horsepower of the motors for the bridge replacement alternative would also be larger to enable one motor operation under all normal loading conditions with the ability to use two motors in the event of extreme ice/snow or wind loads.

5.3.3 Brakes

A primary difference with the machinery for a new bridge would be the braking. Each bascule leaf would be provided with two motor brakes, one located on each of the primary reducer input shafts. Two machinery brakes would also be provided, one on the input shaft of each of the secondary reducers. The motor brakes would provide the torque for stopping the span during operation, while the machinery brakes provide additional braking torque for holding the bridge during high wind loads.

5.3.4 Rear Locks

The rear lock assemblies would be sized to lift the rear portion of the leaf, pushing the rear live load shoes into the uplift girder. This would preload the live load shoes and reduce the impact from traffic loads seen by the uplift girder. The rear lock assembly would consist of a strut with a
rocker assembly at the top, driven into place by a hydraulic cylinder and hydraulic power unit (HPU). Each rear lock assembly would be provided with a HPU, which would have an integrated manual hand pump for emergency operation. When the strut is driven into place, the rocker would travel over center slightly, locking into place while providing uplift on the leaf. See Figure 5-2 for the proposed rear lock assembly.

![Rear Lock Assembly for Replacement Bridge](image)

**Figure 5-2: Rear Lock Assembly for Replacement Bridge**

### 5.4 ELECTRICAL

A double-leaf replacement bascule bridge would be designed to operate under all conditions, including the loss of a power feed or main motor. The new electrical power and control systems, traffic gates, locks and backup power system would provide reliable and redundant systems to operate the bridge for many years.

#### 5.4.1 Traffic Control Devices

New red/yellow/green traffic signals utilizing light emitting diode (LED) bulbs are recommended to reduce operating and maintenance costs. New signals would be designed to meet the current Manual for Uniform Traffic Control Devices (MUTCD).

Four new traffic gates including gongs on the on-coming gates are would be installed on the new structure.

#### 5.4.2 Navigational Signal System

New LED pier lights and span-mounted channel lights would be used for the new bridge alternative. New LED fixtures are cost effective, long lasting, and energy efficient with a 10-year bulb life. New channel floodlights to illuminate the waterway are recommended. In
addition, new bridge communication devices would be installed that include a marine radio, public address system and air horn.

5.4.3 Span Drive System

The new span drive system would be designed using AC induction motors with AC variable speed drives. Under normal operating conditions, the bridge would be designed to operate using only one of the two motors provided with each leaf. In order to meet all AASHTO loading conditions, the provision would also be made to operate each bascule leaf using both motors. That provision would likely only be needed when there is either a heavy snow or ice load on the bridge or when there is a high wind. Using this approach would allow the bridge to be operated normally with redundancy in the event of a motor or drive failure. The new motors and drives would alternate each time the bridge is opened, with the second motor and drive serving as a backup. In the event of a failure of one motor or its electronic drive, the second set would still be available to operate the bascule leaf.

AC induction motors would be provided instead of DC ones primarily because of cost and maintenance considerations. AC motors and drives typically cost less than their DC counterparts and with their current technology, they provide similar speed control characteristics.

5.4.4 Control System

The electrical control system would include an operator console, PLC cabinet and motor control center (MCC). A programmable logic controller (PLC) is recommended as the primary system to control and interlock the operation of the bridge. A backup simple relay control system would also be included as a redundant means of operation in the event of a failure in the PLC system. Using the backup relay system would require more time to perform a bridge opening because it would require a greater number of interventions by the operator in manually activating all of the necessary control buttons and switches.

The operator console would include pushbuttons, switches and indicator lights to provide operator commands to the PLC. In addition, a touch screen display would be used to supplement the feedback from the control system. The touch screen would provide diagnostics and alarms when there is a problem.
5.4.5 Electrical Service

New electrical service would be incorporated with a new generator and automatic transfer switch.

5.4.6 Miscellaneous

New submarine cables and droop cables would be required. All lighting, receptacles, conduit and other equipment would be installed to meet all current codes.

5.5 ROADWAY SECTION

This report considers study limits between 40 feet on either side of the bridge limits. The approach slabs are a standard 20 feet on either side of the bridge limits and therefore roadway considerations are taken to be 20 feet on either side of the approach slabs. Both roadway approaches would need to be reconstructed as part of a bridge replacement. The bridge deck would be widened to accommodate wider shoulders, and therefore the approach roadways would need to be reconstructed to match the typical section of the bridge. Appropriate guardrail transitions and terminations should be provided in accordance with MDOT standards. Based on information available in the existing bridge plans, there appears to be adequate right-of-way to provide the wider approach roadways that would be needed to accommodate the recommended bridge cross section for the replacement alternative.
6 COST ESTIMATES

6.1 BRIDGE REHABILITATION COST ESTIMATE

Estimated construction costs for the bridge rehabilitation alternative are detailed in Appendix A. They are summarized and subtotaled by structural, track and tread, mechanical and electrical related items. Roadway related items are minimal and are incorporated into the structural costs. All costs are escalated at a rate of 5% for each year up to the year 2018, which is the estimated year in which rehabilitation would take place.

6.1.1 Bridge Rehabilitation Structural Costs

Structural rehabilitation costs were developed using the MDOT Weighted Average Item Price Report for the period October 2011 to February 2013 and then escalating them to the Year 2018. Costs for several lump sum items were obtained by referring to recently constructed or rehabilitated movable structures of similar size and type. Quantities were developed by referring to the existing plans.

Due to the high frequency of bridge operations, painting of the bridge would likely need to be performed during the winter months. Due to the increased difficulty in preparing surfaces and applying paint in the cold weather season, the unit price for “Steel Structure, Cleaning and Coating, Partial, Type 4” were increased from those in the MDOT Weighted Average Item Price Report.

The higher unit price of $2.10 per pound for epoxy coated reinforcing steel used in estimating the cost of the bridge rehabilitation alternative compared to the $1.20 per pound used for the replacement alternative reflects the large difference in quantities required between the two alternatives - 4,300 pounds for the rehabilitation versus 461,000 pounds for the replacement. Data from the MDOT Weighted Average Item Price Report for small quantity projects was used as the basis to estimate the average unit price for the rehabilitation alternative and that for large quantity projects for the replacement alternative.

The unit price of $40 per pound for “Structural Steel, Retrofit, Furnish, Fabricate and Erect” for the rehabilitation alternative is also significantly higher than the $6.50 per pound for “Structural Steel” for the replacement alternative, partially due to the large difference in quantity. Additionally, the structural steel work for the rehabilitation alternative consists of installing numerous relatively small pieces in multiple locations with extensive field drilling of holes. This work primarily pertains to making retrofits to the bascule span floor beams on the front arms of the bascule leaves and the transfer girder behind the counterweights on the back arms to address the cracks that have developed near the ends of those members. Access for much of that rehabilitation work will also be difficult with it having to be performed under the bascule leaves over water.
6.1.2 Track and Tread Rehabilitation Costs

Track and Tread rehabilitation costs were developed using historical unit costs of similar items on movable bridge rehabilitation projects to determine material costs and labor rates to perform the repairs specified herein. Additional costs for testing, jacking and support systems are also included in these costs. Due to the additional complexities of fabrication, temporary shoring, and field installation associated with this work, an additional 30% contingency was included in the unit prices for each of the elements comprising this work in its detailed cost estimate in Table A-3 of Appendix A.

6.1.3 Bridge Rehabilitation Mechanical Costs

Mechanical rehabilitation costs were developed using historical costs of machinery on similar rehabilitation projects to determine material costs and labor rates to perform the repairs specified herein. Other assumptions that affected the cost of rehabilitating the operating machinery was access to the machinery items, such as having to remove the floating shafts and couplings to replace the primary reducer output seals and the secondary reducer input seals. Rehabilitation work items with the same name as items in the replacement bridge alternative cost estimate do not necessarily have the same unit costs. This reflects the difference in component sizes and installation effort on a new bridge compared to an existing one.

6.1.4 Bridge Rehabilitation Electrical Costs

Electrical rehabilitation costs were developed using electrical bid item tabulations from two recent movable bridge projects. The work is separated into multiple bid items that include:

- **Electrical Installation** – this item includes conduit, wiring, navigational aids, lighting, limit switches, miscellaneous electrical equipment and installation costs of the electrical contractor.

- **Replace PLC Cabinet & Programming** – This item includes the cost of the PLC and relay equipment and its associated programming.

- **Motor Control Center** – This item includes all costs associated with providing a new motor control center.

- **Console Repairs and Modifications** – This item includes the costs of a replacing and adding new switches and devices to the existing control console in the operators house and providing a new touch screen display.

- **Replace Drives** – This item includes the costs of AC variable speed drives and new motors.

- **Replace Traffic Gates** – This item includes the costs of providing all new traffic gates.

- **Generator Replacement** – This item includes the costs of providing a new generator at a new location, on shore.
• **Electrical Testing, Manuals & Training** – This item includes the cost of all bridge testing, maintenance manuals, spare parts and operator training.

Reference project bid tabulation prices were scaled and adjusted based on the size of the bridge and its equipment. Escalation to the year 2018 was included. Generator costs were developed using data from a separate recent project that included a new 300 KW natural gas generator.

6.1.5 **Bridge Rehabilitation Cost Summary**

Included in the Construction Total is a 15% allowance for un-quantified costs. These costs are typically not computed in a preliminary study due to lack of detail design. Also included is a 20% contingency for unknown and unforeseen issues that would not surface until final design.

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<th>Unit Price</th>
<th>Cost</th>
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**Sub Total** $13,421,100

- Mobilization 5% $671,100
- Allowance for Un-quantified Costs 15% $2,013,200
- Contingencies 20% $2,684,300

**Construction Total** $18,790,000

Table 6-1: Bridge Rehabilitation Cost

6.2 **Bridge Replacement Cost Estimate**

Estimated construction costs for the bridge replacement alternative are detailed in Appendix B. They are summarized and subtotaled by structural, mechanical and electrical related items. Roadway related items are minimal and are incorporated into the structural costs. All costs are escalated at a rate of 5% for each year up to the year 2018, which is the estimated year in which a replacement would take place.

6.2.1 **Bridge Replacement Structural Costs**

Quantities were primarily developed by establishing the preferred geometric layout for a replacement structure and then comparing recently constructed structures of similar size and type. Costs for the structural components of the replacement alternative were then developed using the MDOT Weighted Average Item Price Report for the period October 2011 to February 2013 as well as bid tabulations from similar movable bridge projects.
6.2.2 Bridge Replacement Mechanical Costs

Mechanical replacement costs were developed using historical cost estimates of machinery combined with bid tabulations from recently fabricated movable bridges of relatively similar size and configuration. The sizing of the replacement bridge operating machinery is based on the preliminary configuration of the replacement bridge, as detailed within the structural section. Once a horsepower requirement is determined, the remaining components are chosen based on 1.5 x the motor horsepower, per AASHTO movable bridge specifications.

The rear lock replacement costs were developed using the same historical cost estimates of machinery combined with bid tabulations from recently fabricated movable bridges. The sizing of the rear lock was based on bridges recently built of similar size and roadway configuration.

6.2.3 Bridge Replacement Electrical Costs

Electrical rehabilitation costs were developed using electrical bid item tabulations from two recent movable bridge projects. The work is separated into multiple bid items that include:

- **Electrical Installation** – this item includes conduit, wiring, navigational aids, lighting and miscellaneous electrical equipment and installation costs of the electrical contractor.

- **PLC Cabinet & Programming** – This item includes the cost of the PLC and relay equipment as well as its associated programming.

- **Motor Control Center** – This item includes the costs of a new MCC.

- **Control Console** – This item includes the costs of a new control console and a touch screen display.

- **Motors and Drives** – This item includes the costs of AC variable speed drives and motors.

- **Traffic Gates** – This item includes the costs of all new traffic gates.

- **Generator** – This item includes the costs of a new generator mounted on shore.

- **Limits and Sensors** – This item includes the lock and span limit switches as well as leaf position sensors.

- **Submarine Cables** – This item includes the cost of the submarine cables and their installation.

- **Lightning and Surge Suppression** - This item includes the cost of a bridge lightning protection system and surge protectors on all equipment.

- **Auxiliary Electrical Equipment** – This item includes items such as CCTV, servers and PA systems.
• **Electrical Testing, Manual and Training** – This item includes the cost of all manuals, training and spare parts.

Reference project bid tabulation prices were scaled and adjusted based on the size of the bridge and it’s equipment. Escalation to the year 2018 was included. Generator costs were developed using data from a separate recent project that included a new 300 KW natural gas generator.

### 6.2.4 Replacement Bridge Summary

Included in the overall cost of each project are allowances for un-quantifiable costs. These are costs typically not computed in a preliminary study due to lack of detail design. Also included is a contingency for unknown and unforeseen issues that would surface during final design.

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*Table 6-2: Bridge Replacement Cost*
7 LIFE CYCLE COST ANALYSIS

Life cycle cost analysis (LCCA) is an engineering economic analysis tool that compares all relevant costs that occur throughout the life of multiple alternative solutions. The LCCA process can be separated into five steps.

1. Establish design Alternatives
2. Determine the activity timing
3. Estimate of Costs
4. Compute life cycle costs
5. Analyze the results

Following FHWA guidelines, a life cycle cost analysis (LCCA) was performed to compare the rehabilitation and replacement alternatives. The LCCA took into account the initial capital cost of each alternative in addition to future rehabilitation and/or replacement work that would be required within the analysis period.

It is important to note that the lowest LCC alternative may not always be the best alternative particularly if the alternatives differ in the level of risk or service. Other considerations, outlined in Chapter 8 should also be considered.

Life Cycle Cost Analysis tables are available in Appendix C.

7.1 LCCA Activities and Timing

The LCCA takes into account the initial capital cost of the given alternative in addition to the major rehabilitations or replacement required in the future within the analysis period. The analysis period chosen for the LCCA is 90 years, as this is the typical design life of a new structure (in this case the bascule span and approach spans service life are both estimated to be 90 years). Each LCCA includes multiple rehabilitations for painting, thin epoxy deck overlays and shallow concrete deck overlay, and electrical/mechanical rehabilitations that would be required to maintain the bridge for the next 90 years.

7.1.1 Service Life

The assumed service life describes the approximate time a given structure rehabilitation or new structure would last under normal operating circumstances before another rehabilitation or structure replacement is required. The service life is based upon the service lives that other bridges have been able to achieve with proper maintenance and periodic rehabilitation.

7.1.2 Remaining Service Life Value

If the service life of an alternative extends beyond the end of the analysis period, the remaining service life value is included in the LCCA as a credit to the life-cycle cost. This is the case for the rehabilitation option since the bridge would only be 40 years old at the end of the 90 year
period. The residual value assigned to the rehabilitation alternative is calculated in this analysis as a pro-rated amount based on linear depreciation of the replacement cost of the structural items. The electrical and mechanical items are not included because it is assumed they would be in need of replacement at the end of the analysis period.

### 7.1.3 Bridge Deck Life

Information provided by MDOT was used to determine the useful life of an epoxy coated rebar deck and common deck rehabilitations based on their conditions. It is assumed that a new epoxy coated rebar deck would last approximately 60 years with no maintenance required for the first 25 years. At this point, a thin epoxy overlay would be appropriate and last for approximately 15 years. After the epoxy overlay has served its useful life, a shallow concrete overlay is recommended and assumed to last for 20 years. The end of the 20 year life span of the concrete overlay would coincide with the end of the useful life of the bridge deck and a full replacement would be required.

For the life cycle cost analysis, a new half-filled grid deck is assumed to last 90 years. This assumption is based on experience with similar grid decks on other movable structures. The concrete fill and the deck pans serve to protect a half-filled grid deck and preserve the steel. Throughout the life of the grid deck, epoxy overlays are recommended for preservation.

With the existing superstructure being constructed in 1987, for a rehabilitation alternative the assumption for a 60 year deck life span would result in replacement of the deck at year 35 of the life cycle cost analysis. The LCC assumes that a bridge replacement would occur in year 50 and it is not economical to replace a bridge deck 15 years prior to replacing the structure. Therefore, another round of epoxy overlays is recommended and assumed to carry the bridge decks out to a total life of 75 years at which point a full bridge replacement would occur.

### 7.1.4 Paint System Life

A paint specialist was consulted to assist in assessing the remaining life of the existing paint system as well as give guidance on new paint systems. It was determined that because a partial painting was performed in 2005, the existing paint would last another 25-30 years. The specialist also stated that modern paint systems last 50 years with only minor repairs during that duration.

For the replacement alternative, it is assumed that a modern paint system would be applied and have a useful life of 50 years. To account for minor repairs, the life cycle cost analysis assumes a partial painting 25 years after the new paint system is applied. The approach spans of a replacement bridge would have low maintenance pre-stressed concrete girders and would not require painting.

For the rehabilitation alternative, a partial painting is recommended at year zero. The partial painting would be required because of wear from other rehabilitation recommendations, but also would assist in allowing the paint system to last 25 years after the rehabilitation. This would be
the end of paint useful life as assessed by the paint specialist, at which point a modern paint system would be applied.

7.1.5 Mechanical Systems Life

Mechanical rehabilitations are recommended every 25 years, with a minor rehabilitation at 25 years and a major rehabilitation at 50 years. The minor rehabilitation for the machinery usually consists of replacing seals, gaskets and other rubber parts within the machinery that do not hold up to the environment. The major rehabilitation for the operating machinery usually consists or replacing the seals and gaskets again along with lubrication systems and possibly bearings within the reducer. Brakes are also changed out during a major rehabilitation. Depending on location, and how well they are protected from the environment, the rear locks usually require more extensive repairs at 25 years and replacement at 50 years.

7.1.6 Electrical Systems Life

Electrical rehabilitations are recommended every 25 years. This is largely due to obsolescence of electronic equipment such as Programmable Logic Controllers (PLCs) and variable speed drives. Each rehabilitation would not need to include all equipment, but items such as droop cables, limits, lighting, variable speed drives, PLCs and any other electronic equipment should be replaced every 25 years and all electrical equipment should be replaced after 50 years.

7.2 Costs and Calculations

Rehabilitation and replacement costs are detailed in Chapter 6.

7.2.1 Discount Rate

The discount rate used for the LCCA represents the opportunity cost for the public and reduces future costs to their value in present dollars. A discount rate of 4% is used in this LCCA.

7.2.2 Present Value

The LCCA compares all costs using the present value of money. 2018 dollars would be used for both the rehabilitation and the replacement comparison. The following formula calculates the present value of expenditure:

\[
PV = \frac{1}{(1 + i)^n} \]

where:

\[
PV = \text{Present Value of expenditure}
\]

\[
Initial\ Cost = \text{Cost of activity in real dollars}
\]

\[
i = \text{Discount rate}
\]

\[
n = \text{Year of expenditure}
\]
The Net Present Value (NPV) is calculated by summing the present value of all cost component activities.

### 7.2.3 Annual Maintenance Costs

If evaluating for Equivalent Uniform Annual Cost (EUAC), the annual maintenance costs can be added to the EUAC. However, if the annual costs are different at different periods of the life-cycle analysis, then the annual costs can be converted to a single present sum, using the Uniform Series Present Worth (USPW) equation at the beginning of the period for disparate annual costs:

\[
USPW = AC \frac{(1 + i)^n - 1}{i(1 + i)^n}
\]

where:
- \(USPW\) = Uniform Series Present Worth
- \(AC\) = Annual Cost
- \(i\) = Discount rate
- \(n\) = Duration of annual cost

The resulting USPW can then be converted to present value, using the Present Value equation.

### 7.2.4 Equivalent Uniform Annual Costs

Once a net present value (NPV) has been determined, all costs within the analysis period are prorated to an annual basis. This is achieved through the calculation of the Equivalent Uniform Annual Cost (EUAC):

\[
EUAC = NPV \frac{i(1 + i)^n}{(1 + i)^n - 1}
\]

where:
- \(EUAC\) = Equivalent Uniform Annual Cost
- \(NPV\) = Net Present Value
- \(i\) = Discount rate
- \(n\) = Number of years into future

While annual maintenance costs are not accounted for in this analysis, they can be added to the EUAC to produce a Total Annual Cost for the alternative.
7.3 LIFE CYCLE COST ANALYSIS ASSUMPTIONS

The following assumptions were used to develop the LCCA:

- Rehabilitation Year = 2018
- Replacement Year = 2018
- Replacement design life and service life (bascule span) = 90 years
- Replacement design life and service life (Approach Spans) = 90 years
- Existing structure design life and service life = 50 years beyond year zero
- Reinforced concrete bridge deck service life = 60 years
- Steel grid bridge deck configuration total service life (bascule span) = 90 years
- Bridge deck service life before first overlay (All Spans) = 25 years
- Life of bridge deck repairs:
  - Thin Epoxy Overlay = 15 years
  - Shallow Concrete Overlay = 20 years
- Paint systems for structural steel are assumed to last for 50 years with a partial painting performed every 25 years
- Mechanical and Electrical repairs and/or rehabilitations are required every 25 years
- Annual Maintenance Costs are assumed to be approximately $42,117 in 2018 dollars for the bridge rehabilitation alternative and $33,000 in 2018 dollars for the bridge replacement alternative. These costs include inspection efforts and routine/preventative maintenance.
- Bridge operation costs are assumed the same for each alternative and are therefore are neglected.
- Highway user delay costs are neglected
- No cost is factored in for obsolescence
- Analysis period = 90 years
- Discount rate = 4%
- Real Dollars = 2018 Dollars

7.4 LCCA Results

Tables 7-1 and 7-2, below, summarize the results of the LCCA for a 90-year analysis period, using a discount rate of 4.0%.
### Table 7-1: LCCA Summary—Rehabilitation Alternative

<table>
<thead>
<tr>
<th>Cost Component Activity</th>
<th>Year</th>
<th>Cost</th>
<th>Present Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rehabilitation</td>
<td>0</td>
<td>$18,790,000</td>
<td>$18,790,000</td>
</tr>
<tr>
<td>Thin Epoxy Deck Overlay (Bascule Span)</td>
<td>15</td>
<td>$40,400</td>
<td>$22,433</td>
</tr>
<tr>
<td>Shallow Concrete Overlay (Approach Spans)</td>
<td>15</td>
<td>$199,100</td>
<td>$110,553</td>
</tr>
<tr>
<td>Replace Approach Slabs</td>
<td>15</td>
<td>$24,900</td>
<td>$13,826</td>
</tr>
<tr>
<td>Joint Repairs</td>
<td>15</td>
<td>$49,200</td>
<td>$27,319</td>
</tr>
<tr>
<td>Mechanical &amp; Electrical</td>
<td>25</td>
<td>$2,518,400</td>
<td>$944,694</td>
</tr>
<tr>
<td>Steel Structure, Cleaning and Coating, Full, Type 4</td>
<td>25</td>
<td>$662,400</td>
<td>$248,477</td>
</tr>
<tr>
<td>Operator House (Roof, Doors, Windows)</td>
<td>25</td>
<td>$96,800</td>
<td>$36,311</td>
</tr>
<tr>
<td>Thin Epoxy Deck Overlay (Bascule Span and Approach Spans)</td>
<td>35</td>
<td>$81,800</td>
<td>$20,729</td>
</tr>
<tr>
<td>Joint Repairs</td>
<td>35</td>
<td>$49,200</td>
<td>$12,468</td>
</tr>
<tr>
<td>Bridge Replacement</td>
<td>50</td>
<td>$45,392,000</td>
<td>$6,387,227</td>
</tr>
<tr>
<td>Thin Epoxy Deck Overlay (App. Spans and Bascule Span)</td>
<td>75</td>
<td>$88,800</td>
<td>$4,687</td>
</tr>
<tr>
<td>Joint Repairs</td>
<td>75</td>
<td>$61,200</td>
<td>$3,230</td>
</tr>
<tr>
<td>Steel Structure, Cleaning and Coating, Partial, Type 4</td>
<td>75</td>
<td>$233,100</td>
<td>$12,304</td>
</tr>
<tr>
<td>Operator House (Roof, Doors, Windows)</td>
<td>75</td>
<td>$1,260,700</td>
<td>$66,544</td>
</tr>
<tr>
<td>Thin Epoxy Deck Overlay (Bascule Span)</td>
<td>90</td>
<td>$61,000</td>
<td>$1,788</td>
</tr>
<tr>
<td>Shallow Concrete Overlay (Approach Spans)</td>
<td>90</td>
<td>$246,600</td>
<td>$7,228</td>
</tr>
<tr>
<td>Replace Approach Slabs</td>
<td>90</td>
<td>$24,900</td>
<td>$730</td>
</tr>
<tr>
<td>Joint Repairs</td>
<td>90</td>
<td>$61,200</td>
<td>$1,794</td>
</tr>
<tr>
<td>Replace Coating for Steel Sidewalks (Bascule Span)</td>
<td>90</td>
<td>$58,900</td>
<td>$1,726</td>
</tr>
<tr>
<td>*Remaining Service Life Value</td>
<td>90</td>
<td>$(16,018,000)</td>
<td>$(469,470)</td>
</tr>
<tr>
<td>Annual Maintenance (Year 0 to 90)</td>
<td>90</td>
<td>$42,117</td>
<td>$1,022,072</td>
</tr>
</tbody>
</table>

**Net Present Value (NPV)**: $27,266,671

**Equivalent Uniform Annual Costs (EUAC)**: $1,123,598

* Remaining Service Life Value is calculated based on a percentage of the Structural cost of the replacement bridge. (50yrs/90yrs)($25,070,900)(1.15 contingency) = $16,018,000.
Table 7-2: LCCA Summary—Replacement Alternative

<table>
<thead>
<tr>
<th>Cost Component Activity</th>
<th>Year</th>
<th>Cost</th>
<th>Present Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Replacement</td>
<td>0</td>
<td>$45,392,000</td>
<td>$45,392,000</td>
</tr>
<tr>
<td>Steel Structure, Cleaning and Coating, Partial, Type 4</td>
<td>25</td>
<td>$233,100</td>
<td>$87,440</td>
</tr>
<tr>
<td>Thin Epoxy Deck Overlay (Bascule Span and Approach Spans)</td>
<td>25</td>
<td>$88,800</td>
<td>$33,310</td>
</tr>
<tr>
<td>Joint Repairs</td>
<td>25</td>
<td>$61,200</td>
<td>$22,957</td>
</tr>
<tr>
<td>Operator House (Misc. Work)</td>
<td>25</td>
<td>$15,000</td>
<td>$5,627</td>
</tr>
<tr>
<td>Mechanical &amp; Electrical</td>
<td>25</td>
<td>$1,292,400</td>
<td>$484,801</td>
</tr>
<tr>
<td>Thin Epoxy Deck Overlay (Bascule Span)</td>
<td>40</td>
<td>$61,000</td>
<td>$12,706</td>
</tr>
<tr>
<td>Shallow Concrete Overlay (Approach Spans)</td>
<td>40</td>
<td>$246,600</td>
<td>$51,364</td>
</tr>
<tr>
<td>Joint Repairs</td>
<td>40</td>
<td>$61,200</td>
<td>$12,747</td>
</tr>
<tr>
<td>Replace Approach Slabs</td>
<td>40</td>
<td>$24,900</td>
<td>$5,186</td>
</tr>
<tr>
<td>Replace Coating for Steel Sidewalks (Bascule Span)</td>
<td>40</td>
<td>$58,900</td>
<td>$12,268</td>
</tr>
<tr>
<td>Steel Structure, Cleaning and Coating, Full, Type 4</td>
<td>50</td>
<td>$558,318</td>
<td>$78,562</td>
</tr>
<tr>
<td>Mechanical &amp; Electrical</td>
<td>50</td>
<td>$2,762,300</td>
<td>$388,690</td>
</tr>
<tr>
<td>Operator House (Roof, Doors, Windows)</td>
<td>50</td>
<td>$96,800</td>
<td>$13,621</td>
</tr>
<tr>
<td>Vessel Collision System Repairs</td>
<td>50</td>
<td>$175,100</td>
<td>$24,639</td>
</tr>
<tr>
<td>Thin Epoxy Deck Overlay (Bascule Span)</td>
<td>60</td>
<td>$61,000</td>
<td>$5,799</td>
</tr>
<tr>
<td>Reinforced Conc. Deck Replacement (Approach Spans)</td>
<td>60</td>
<td>$696,900</td>
<td>$66,248</td>
</tr>
<tr>
<td>Replace Concrete Deck (Movable Span over Machinery)</td>
<td>60</td>
<td>$252,200</td>
<td>$23,974</td>
</tr>
<tr>
<td>Replace Approach Slabs</td>
<td>60</td>
<td>$24,900</td>
<td>$2,367</td>
</tr>
<tr>
<td>Remove and Replace Steel Sidewalks (Bascule)</td>
<td>60</td>
<td>$117,700</td>
<td>$11,189</td>
</tr>
<tr>
<td>Steel Structure, Cleaning and Coating, Partial, Type 4</td>
<td>75</td>
<td>$233,100</td>
<td>$12,304</td>
</tr>
<tr>
<td>Mechanical &amp; Electrical</td>
<td>75</td>
<td>$1,292,400</td>
<td>$68,218</td>
</tr>
<tr>
<td>Remaining Service Life Value</td>
<td>90</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Annual Maintenance (Year 0 to 90)</td>
<td>90</td>
<td>$33,000</td>
<td>$800,820</td>
</tr>
<tr>
<td>Net Present Value (NPV)</td>
<td></td>
<td>$47,616,837</td>
<td></td>
</tr>
<tr>
<td>Equivalent Uniform Annual Costs (EUAC)</td>
<td></td>
<td>$1,962,183</td>
<td></td>
</tr>
</tbody>
</table>
8 EVALUATION OF ALTERNATIVES

The existing bridge continues to provide functional service but its bascule span would require an extensive rehabilitation to remain reliable for an extended period. That work would include a replacement of the track and tread assemblies, a new more robust fender system, extensive electrical rehabilitation and numerous structural and mechanical repairs.

As an alternative to a major rehabilitation, the bridge could be replaced with a new one that would include a new double-leaf bascule span. A high-level replacement bridge without a movable span would not be appropriate because of the magnitude of profile raise it would require to accommodate navigation and the associated impacts such a raise would have on the local street system and access to nearby businesses, residences and park facilities. A single leaf bascule structure would not be feasible due to the extensive length it would need to span the navigation channel. A new rolling lift type bascule bridge is preferable to a trunnion style structure because it requires a shorter and therefore more economical superstructure to span the same width of navigation channel. A rolling lift bascule structure also requires less maintenance because Scherzer type center locks without mechanical parts can be utilized to connect the two movable leaves when they are mated.

8.1 EVALUATION CRITERIA

The following table provides criteria for evaluating and comparing the benefits, costs and impacts of the rehabilitation and replacement alternatives:

Table 8-1: Evaluation Criteria

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Measure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Construction Cost</td>
<td>Estimated construction cost</td>
</tr>
<tr>
<td>2 Life Cycle Cost</td>
<td>Total annualized cost of construction and future maintenance</td>
</tr>
<tr>
<td>3 Functionality</td>
<td>Conformance with geometric and safety standards and convenience to users</td>
</tr>
<tr>
<td>4 Long Term Reliability</td>
<td>Likelihood for need of unexpected repairs in future</td>
</tr>
<tr>
<td>5 Risk</td>
<td>Potential for unforeseen issues</td>
</tr>
</tbody>
</table>
These evaluation criteria are further explained below, and rehabilitation and replacement alternatives are rated with respect to them.

### 8.1.1 Construction Cost

The Construction Cost criterion considers the capital expenditure for the initial construction project of a major bridge rehabilitation and a complete bridge replacement. It does not account for additional future actions that would be needed for each alternative. Appendix A itemizes construction costs for the bridge rehabilitation alternative and Appendix B for the bridge replacement. The scope of the work for preparing these estimates is separated into four primary categories: Electrical, Mechanical, Structural and Track/Tread Replacement (for rehabilitation only). Included in the overall cost of each alternative are allowances for un-quantifiable costs. These are costs of miscellaneous work items not able to be readily computed during a feasibility study because detailed design has not yet been performed. Also included is an allowance for contingency for unknown and unforeseen items that often do not become apparent until preliminary engineering or final design.

The estimated construction cost for the rehabilitation alternative is $18,790,000. The estimated cost for the replacement alternative is $45,392,000.

**Electrical:** The estimated cost for electrical work for the bridge rehabilitation alternative is significant. However with some of the electrical equipment still having reliable remaining service life this estimated cost would not be as great as that for a complete bridge replacement.

**Mechanical:** Much of the bridge’s mechanical equipment has reliable remaining service life. Therefore the estimated cost for mechanical work for the bridge-rehabilitation alternative is approximately one tenth that for the bridge-replacement alternative. A new bridge would require two complete new mechanical systems – one for each bascule leaf.

**Structural:** The estimated cost for structural work for the bridge-replacement alternative is substantially more than that for rehabilitation. This difference reflects the greater amount of work to be performed in fully removing and replacing the existing bridge including its movable span and two flanking fixed approach spans with the bridge versus repairing and maintaining the structure for rehabilitation.

The costs are based on a concept-level consideration of the primary elements of work scope for both alternatives. Because geotechnical exploration was not performed as part of this study,
information from existing plans was used as the basis in formulating an assumed foundation system for the replacement alternative and its associated cost.

8.1.2 Life Cycle Cost

This criterion considers the overall costs of providing and maintaining a bridge for an extended planning horizon. It takes into account the initial capital cost of each alternative plus all additional costs that are anticipated over the life of each alternative. Those additional costs include the major rehabilitations or replacement required over the analysis period. 90 years was assumed for the analysis period because this is a good approximation of the design life of a new structure of this type.

The life cycle cost takes into account the present value of money. A 4% discount rate was used for the life cycle cost analysis. The Net Present Value of each alternative was obtained by summing the present value of all initial and future cost component activities.

The cost of the rehabilitation alternative has a lower net present value, as is often the case with these types of studies. This difference is primarily due to the significantly larger initial cost of the bridge replacement alternative.

8.1.3 Functionality

This criterion addresses how effectively each alternative accommodates bridge users. The bridge rehabilitation alternative would maintain the existing cross section geometrics, including its 30-foot curb-to-curb width, which accommodates one 12’-0” traffic lane in each direction and 3’-0” shoulders. With this narrow curb-to-curb width, use of the roadway by bicyclists is undesirable.

The bridge replacement alternative would provide a 40-foot curb-to-curb width that would accommodate one 12’-0” traffic lane in each direction plus a center channelization lane for left hand turns at the intersections just beyond both ends of the bridge and 2’-0” shoulders. The bridge replacement alternative would provide 5-foot sidewalks on both sides of the roadway, similar to the existing bridge.

Both alternatives would maintain the 150-foot horizontal clearance for navigation. When in the closed position, the movable span for both alternatives would also maintain the 20-foot vertical clearance for smaller boats that do not require bridge openings and associated traffic disruptions.

8.1.4 Long Term Reliability

The existing bridge’s substructures were built in 1938. Overall, they remain structurally sound. However, they would require surface repairs and other similar maintenance to continue to function reliably for an extended period of time. Based on the rehabilitation alternative extending the useful life of the bridge by another 50 years, those substructures would be 130 years old before they would be replaced as part of a new bridge. Testing of concrete core samples indicates the bascule pier concrete has good strength and physical properties. However, the substructures will be subject to deterioration as they continue to age. Foundation loading
calculations indicate the timber piles that support the bascule piers are not overloaded; however their condition cannot be directly observed to be readily assessed.

The upper portions of the timber piles supporting the bascule piers are encased in 5-foot thick concrete seals and terminate within the bottom of 4-foot thick reinforced concrete footings. As such, they are not accessible to take samples to test for deterioration. However, these timber piles likely remain in sound condition. The surrounding concrete of the footings and seals has provided them protection from physical damage. Because the top six feet of these piles is encased in concrete and their remaining portions are more than 25 feet below the water surface, they have also been protected from any exposure to air that could lead to deterioration. Similar timber pile systems whose conditions were observed during foundation removal as part of geometrically deficient movable bridge replacement projects have been observed to be in good condition. There have been no indications of movement by the bascule piers due to settlement or displacement of the timber piles.

There is a history of timber pile supported bascule bridges having lifespans well beyond 100 years with major rehabilitations being performed on them with the expectation of their foundation systems lasting well beyond that age. A major rehabilitation was recently completed in 2013 of the Michigan Street Rolling Lift Bascule Bridge in Sturgeon Bay, Wisconsin. That bridge is 84 years old. In 1997, a major rehabilitation was performed on the 108 year old Cermak Road Rolling Lift Bascule Bridge over the South Branch of the Chicago River. Both of those structures are supported on timber piles in a similar fashion as the Lafayette Avenue Bridge.

Practical advanced NDE techniques for sampling and testing the physical condition of inaccessible timber piles like the ones on this bridge are not available. A precision survey could be performed on the bascule piers to help provide confidence in the current load carrying capability of the bascule piers’ timber pile systems. A key attribute of a rolling lift bascule bridge is that the entire weight of its bascule leaves including their massive concrete counterweights shifts on the supporting pier as each leaf rolls back and forth. This shifting heavy load causes a significant change in the loading on the foremost and backmost rows of piles during bridge openings. Using precision survey equipment, the front and back walls of the bascule piers could be monitored during several test bridge openings to confirm that no significant substructure movement occurs due to inadequate support by the timber pile system.

Based on the assumption that the timber pile foundation system remains sound and considering the results from testing of concrete core samples that were taken from the areas of the bascule piers most susceptible to the elements, a remaining life of 50 years has been assumed in the life cycle cost analysis for the bridge rehabilitation alternative. The sensitivity of this assumption was investigated by calculating the life cycle costs for the scenario where the bridge has to be replaced 30 years earlier due to unforeseen conditions (i.e. considering only 20 years of remaining life after the rehabilitation). This would be the time that the substructures and supporting piles are 100 years old rather than 130. Moving the schedule for bridge replacement
up 30 years significantly effects the life cycle costs by increasing the Net Present Value by approximately $13,900,000 and increasing the Equivalent Uniform Annual Costs by approximately $570,000.

The existing bridge is classified as scour critical. Scour countermeasures have been performed at the bases of the bascule piers in the past and additional scour remediation efforts are recommended for the rehabilitation alternative at the downstream ends of the piers. Although there is currently no undermining of the bascule piers, additional countermeasures may be needed over the next 50 years.

8.1.5 Risk

This criterion provides a means to consider and compare the potential for unanticipated issues during implementation of each alternative.

The challenging and complex work of temporarily lifting and shoring each massive bascule leaf and then reconstructing to machine tolerances its geometrically complex tread and track assemblies as part of the rehabilitation alternative has the potential for unforeseen challenges. This type of work is not routinely performed on bascule bridges.

Because the precise condition of every element of a movable bridge of this vintage cannot be always verified or known in advance, the potential for construction change orders to perform additional work during construction is also greater for the rehabilitation alternative.

8.1.6 Constructability

This criterion compares the constructability of the rehabilitation and the replacement alternatives. The replacement bridge alternative would require removing the entire structure and replacing it in the same location. This alternative may be more inconvenient to bridge users because of longer construction time, but does not appear to have any unique constructability issues.

The rehabilitation alternative would have complex and difficult repairs to the tracks and treads of both leaves. These repairs would require the closure of half the waterway, which may require the work to be completed from January to March during which the bridge is normally closed to marine traffic. This is a tight time window. The work would also require driving piles near the fenders to support jacking of the bascule leaves.

8.1.7 Construction Disruption

This criterion considers the extent of disruption to users of the bridge during construction.

Both alternatives would require a closure of the bridge to vehicular and pedestrian traffic and the need for a detour route. Bridge rehabilitation would require a closure period of approximately 9 months. With its greater scope of work including the lengthy process of removing existing bascule substructures and constructing new ones, bridge replacement would require a longer closure period of approximately 21 months. The estimated closure time for the bridge
replacement alternative also takes into account the potential use of a limited accelerated bridge construction method (ABC) that is discussed further below.

Based on US Coast Guard requirements, construction disruptions other than very brief ones to mariners for either alternative will not be allowed. Therefore, both alternatives would need to avoid work that would encroach into the navigation channel during the navigation season. For the rehabilitation alternative, replacing the track and tread assemblies with the bascule leaves in the lowered position over the navigation channel would need to be performed during the non-navigation season over the winter months.

For the replacement alternative, construction of the portion of a new bascule superstructure over the navigation channel could be performed in several ways. The work could be performed over the winter months with the front arms of the bascule leaves erected in the closed position and extending over the navigation channel. Alternately, the front arms could be erected during the regular construction season with the bascule leaves secured in the open position to not inhibit navigation.

Another potential method for constructing the front arms of the bascule leaves during the regular construction season would be to pre-assemble them offsite, float them in, and then lift them into final position with the bridge in the closed position. This method of ABC construction technique has been effectively used for some bascule bridge projects when extended navigation disruptions were not permitted and erection in the open position was considered by the contractor to be undesirable for construction duration, safety, cost, and/or other concerns. With this ABC method, once the new bascule piers were constructed, the heel sections of the bascule leaves would then be erected in the closed position within the new piers along with each leaf’s concrete counterweight and on-board operating machinery. Concurrently with that work, the front arms of each leaf would be assembled nearby on barge-mounted temporary shoring. During a brief navigation outage, the pre-assembled front arms would then be floated to the site, hoisted into position, and attached to the previously on-site erected heel sections through bolted slices in the bascule girders. Figure 8-1 provides an example of this ABC method. Though it would have the potential to provide some limited savings in overall construction duration, it would not reduce the lengthy amount of time initially required to remove the existing bascule bascule piers and construct the new ones.
# 8.2 Evaluation Summary

Table 8-2 summarizes the comparison of the rehabilitation and replacement alternatives against the evaluation criteria. Red cells indicate criteria on which the alternative is least desirable; yellow indicates moderate impacts, and green indicates positive performance on the criterion.

**Table 8-2: Evaluation Matrix**

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Rehabilitation</th>
<th>Replacement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction Cost</td>
<td>Lower Initial Cost = $18,790,000</td>
<td>Higher Initial Cost = $43,392,000</td>
</tr>
<tr>
<td>Life Cycle Costs</td>
<td>Net Present Value = $27,266,671</td>
<td>Net Present Value = $47,616,837</td>
</tr>
<tr>
<td></td>
<td>Equiv. Uniform Annual Cost = $1,123,598</td>
<td>Equiv. Uniform Annual Cost = $1,962,183</td>
</tr>
<tr>
<td>Functionality</td>
<td>Remains the same</td>
<td>Wider roadway deck promotes safety &amp; accommodates center channelization lane for left turns</td>
</tr>
<tr>
<td>Long Term Reliability</td>
<td>Substructures would be 130 years old before bridge is replaced</td>
<td>New bridge built to current codes and requirements Scour resistant Substructure</td>
</tr>
<tr>
<td></td>
<td>Additional future scour countermeasures likely required</td>
<td></td>
</tr>
<tr>
<td>Risk</td>
<td>Greater potential for unforeseen issues with major structural repairs</td>
<td>Fewer unknowns with all-new construction Ability to fully considerer potential issues in new design</td>
</tr>
<tr>
<td></td>
<td>Higher likelihood for possible issues with 80-year old substructures</td>
<td></td>
</tr>
<tr>
<td>Constructability</td>
<td>Specialized &amp; complex repairs for track and tread replacements</td>
<td>Typical Movable Bridge Construction</td>
</tr>
<tr>
<td></td>
<td>Jacking and shoring leaves Major work during winter</td>
<td></td>
</tr>
<tr>
<td>Construction Disruption</td>
<td>9-Month Roadway Closure</td>
<td>21-Month Roadway Closure</td>
</tr>
</tbody>
</table>

**Matrix Key**

<table>
<thead>
<tr>
<th>Relative Comparison</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Large Impact – Less Desirable</td>
<td></td>
</tr>
<tr>
<td>Moderate Impact</td>
<td></td>
</tr>
<tr>
<td>Small Impact – More Desirable</td>
<td></td>
</tr>
</tbody>
</table>
Some of these criteria have been evaluated qualitatively. They may not be considered of equal importance to all stakeholders. However, the evaluation matrix provides a summary of the costs, benefits and impacts of the alternatives.

Concepts and details for alternatives of bridge rehabilitation and bridge replacement were developed, evaluated and compared. The initial construction cost and life cycle cost of bridge rehabilitation is less than the bridge replacement alternative. Bridge rehabilitation would require a shorter period of bridge closure for initial construction but would be less effective in improving geometrics or accommodations for non-motorized traffic. Although the initial construction disruption would be less for a rehabilitation project, over its lifecycle the total disruptions for a new bridge would be less due to its inherent superior strength and durability.

The work associated with replacement of the track and tread assemblies is extensive and complex. Detailed repair plans and specifications would be required to enable a competent qualified contractor to perform the work. Although a replacement rolling lift bascule bridge would have a similar arrangement of track and tread assemblies, those components could be fully assembled, tested and verified in the controlled environment of a machine shop using state of the art equipment prior to installation in the field.