



REHABILITATION STUDY REPORT

Submitted by Close, Jensen & Miller, P.C.

BRIDGE NO. 01349

Route 136 over Saugatuck River

Town of Westport



Issued June 2016

State Project No. 0158-212

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TOWN OF WESTPORT

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EXECUTIVE SUMMARY

Purpose of the Project:

The project was initiated to address the structural and functional deficiencies of Bridge No. 01349, carrying Route 136 over the Saugatuck River in the Town of Westport. The truss continues to see collision damage due to the deficient width of the bridge for the high volume of traffic crossing the bridge. Portions of the original substructure that were repaired in the 1950's now need reconstruction to continue to support the loads under current requirements. The purpose and need is to change the conditions to bring the structure to a state of good repair, fix the functional issues that are causing continuous damage to the trusses, and be sensitive to the historical importance of the structure.

Current Bridge Deficiencies:

The existing bridge exhibits the following structural and functional deficiencies:

Existing Truss System:

The existing truss system is in critical condition. During the bridge replacement performed in 1993, the trusses were retained for ornamental purposes only and carry no live load. While the trusses are no longer considered fracture critical to the superstructure, they remain fracture critical by design; and as such, failure to a primary truss member, if not properly maintained, could risk truss collapse and pose a safety hazard to vehicles and pedestrians. The trusses have sustained severe and consistent vehicular impact damage throughout the entire structure. The wind load capacities for the truss are also substandard.

Deteriorated Condition of Pier 2 piles and cross bracing:

The load capacity of the bridge is substandard, because the Pier 2 support system observed in both the as-built (1951) condition and current deteriorated condition does not meet the current code criteria based on today's standards. The support system piles exhibit substandard Capacity to Demand ratios. The steel diagonal cross-bracing members at Pier 2 have heavy rust and up to 100 percent section loss and are rated as critical. Five of the six piles have loose or missing fiberglass jackets, and the exposed concrete is scaled and exhibits exposed reinforcement.

Bridge Geometry:

Based upon the guidelines put forth in the "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges", the deck geometry is rated a "2", which is classified as "basically intolerable requiring high priority of replacement". The rating comes from the comparison of the Average Daily Traffic (ADT) of 13,100 vehicles in relation to the 19'-6" curb-to-curb width of the roadway along the bridge. The very low rating is an indication that the existing bridge is not functionally capable of handling existing traffic demands. The existing vertical clearance is 12'-10", which is also substandard from the required 14'-0".

Accident History:

The substandard horizontal roadway width is contributing to the high frequency of accidents that occur on the existing bridge structure. There were 16 reported accidents on the bridge and its approaches from 2010-2014. 69% of the recorded accidents are sideswipes, which may be attributed to the narrow lanes on the bridge. The intersections of Route 136 and Riverside Avenue as well as Route 136 and the commercial driveway just west of the bridge are also locations of high accident frequency.

Substandard Guiderail System:

The guiderail system across the bridge is substandard. The existing system is not crash-test compliant. The existing substandard railing, narrow roadway width, and the proximity of the truss system to the edge of the roadway are all contributing to the extensive vehicular impact damage to the truss members. It should also be noted that when

impacted, guiderail is designed to deflect. Because the existing guiderail is located against the truss, the truss is not protected by this type of system.

Mechanical and Electrical System:

The mechanical and electrical equipment, while currently in good condition, are susceptible to damage during flood event of a 10-year storm frequency or higher. The equipment beneath the bridge is positioned between the elevations of 6.0 and 8.4 feet. The 10-year design storm flood elevation is approximately 8.1 feet, resulting in the equipment being nearly submerged during such an event. This occurred during Hurricane Sandy in 2012 and required significant repairs and expense. These events will continue to occur unless the structure is raised above the design flood level.

East Coast Greenway:

Route 136, along the structure, is a part of the East Coast Greenway and supports added pedestrian and bicycle traffic. Currently, the cyclists are forced to use traffic lanes across the bridge due to the lack of proper shoulder widths, thereby creating unsafe conditions for bicycle traffic.

Alternates Studied in the RSR:

A number of alternatives were investigated as a part of the RSR and are as listed below:

No Action

This alternate was dismissed, because it did not address the public safety concerns which cannot be ignored. This alternative would not address the extensively damaged truss elements or the substandard Pier 2 load carrying capacity based upon today's standards.

Minor Repairs

This option involves repair of the damaged and deteriorated truss system in its current position, repairs to Pier 2, and other minor repairs throughout the structure. This alternative was also dismissed because it does not incorporate preventative measures to preclude further truss vehicular damage and does not address the substandard load capacity of Pier 2 based upon today's standards.

One-way Bridge Access

This alternative involves repairing the damaged and deteriorated truss system in its current position, repairs to Pier 2, and replacing the existing bridge railing with a crash compliant system set in order to provide adequate horizontal clearance to preclude impact damage to the trusses. The new railing system would further reduce the existing 19'-6" curb-to-curb width, reducing capacity to one lane of traffic. This alternative was not deemed viable due to the negative impact the redirection of vehicular traffic would cause along the resultant permanent detour route, which is already at or beyond capacity.

Major Rehabilitation

This alternate consists of remediating only the bridge's major structural issues:

- Repair, painting and widening of the ornamental trusses
- Installation of a new bridge railing system to preclude vehicular impact damage
- Reconstruction of the Pier 2 support system
- Increasing the vertical clearance along the bridge to 14'-3" to satisfy functional adequacy for vertical clearance
- Addition of a moment connection to the trusses at panel points to resist wind load stresses
- Deck patching, membrane waterproofing, and paving
- Complete painting of the superstructure steel
- Installation of a solid roadway barrier at both approaches
- Structural steel repairs
- Substructure patching

The bridge would need to be closed to traffic to perform the required rehabilitation measures due to the narrowness of the structure and the significantly deteriorated condition of the truss and pier. The estimated construction schedule for performing the rehabilitation work would be 2.5 to 3 years, and would require construction of a temporary bridge to maintain traffic along Route 136. A detour option was investigated but was determined to provide an unacceptable level of service for the required construction duration. The cost of the major rehabilitation option, including the temporary bridge, is estimated at \$19.8 million.

Structure Replacement

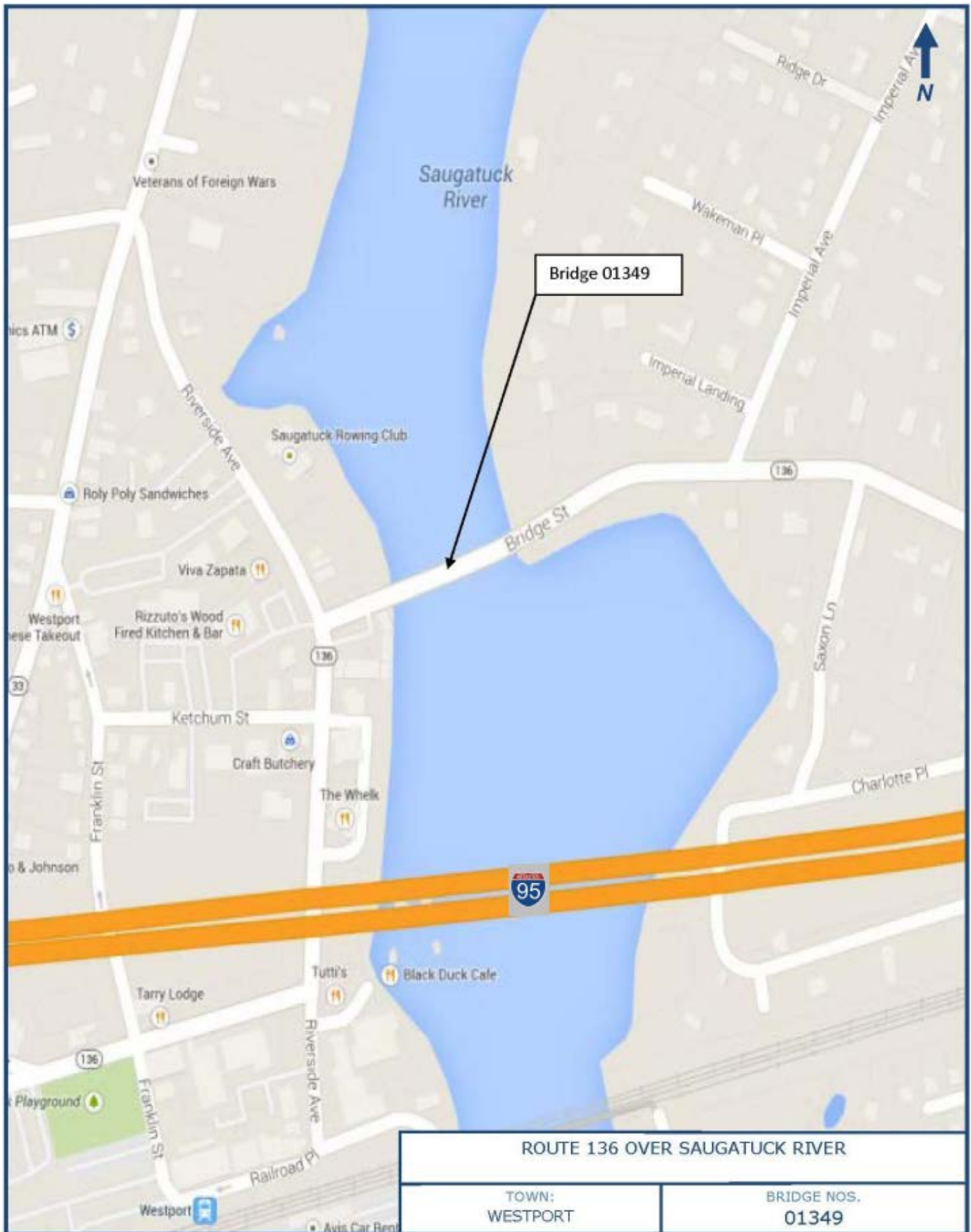
A complete bridge replacement was also investigated as a base line cost comparison for the Major Rehabilitation alternate, which also involved a life cycle cost analysis. A conceptual replacement bridge option would consist of a new four span structure with two multi-girder fixed spans and a two span Pratt Truss swing span. The cost for a new structure was estimated to be \$35.8 million.

A lifecycle cost analysis was performed to estimate the overall cost of each alternative over a span of 75 years and represents costs in today's dollar amounts. The life cycle cost is \$41.27 million for Major Rehabilitation and \$41.43 million for a Structure Replacement.

Design Recommendation:

The No Action alternative does not resolve any of the issues. The Minor Repairs alternate retains the historic character but does not resolve the continued structural damage that occurs to the truss. The One Way Bridge alternate resolves the ongoing bridge damage and retains the historic character, but causes un-resolvable traffic problems for vehicles and bicycles.

The two options that go a long way to address the purpose and need are the Major Rehabilitation alternate and the Structure Replacement alternate. The lifecycle costs for both options are essentially the same and, as such, it is recommended that further in-depth studies of various bridge replacement alternatives be completed before any determinations can be made whether to rehabilitate the bridge or to replace it. A more in-depth benefit/cost analysis is also warranted to take into consideration the cost of lost commuter time due to traffic queues as well as the environmental impacts associated with fuel emissions resulting from idling vehicles. It should also be noted, that in addition to the high initial cost of the major rehabilitation, a number of significant functional deficiencies would still remain unaddressed with a major rehabilitation. These include the geometric deficiencies of the existing bridge for handling the volume of traffic, continued susceptibility of damage to mechanical and electrical features of the structure due to 10-year or greater storm events, and the notable fact that Route 136 is part of the East Coast Greenway, and the substandard width is not acceptable for safe passage for future bikeway and pedestrian needs. Further engagement of all affected stakeholders would be initiated in a public outreach process for developing and assessing bridge replacement alternatives before any determination is made for bridge rehabilitation or replacement.



INTRODUCTION

Close, Jensen and Miller, P.C. has been retained by the Connecticut Department of Transportation (Department) to perform the rehabilitation evaluation for this bridge as part of the State Bridge Program. A compilation of inspection observations, pertinent bridge data and conceptual design plans are included. The report includes detailed inspections of the structure performed by the Department. The recommendations presented have been developed after careful appraisal of the existing structure to ensure the long-term structural and functional adequacy of this crossing. This report considers the safety, serviceability, economics, historical significance and aesthetics of the structure, and serves as the vehicle by which the design and rehabilitation shall be implemented.

DESCRIPTION

General

Bridge No. 01349, built in 1884 and reconstructed in 1993, is a continuous steel multi-girder swing span and fixed span bridge that carries one lane of Route 136 traffic in each direction over the Saugatuck River in Westport. The structure is located between Interchanges 17 and 18 of I-95.

Geometry and Design

Bridge No. 01349, also known as the William F. Cribari Memorial Bridge, is a four span steel multi-girder bridge with an ornamental truss and swing spans that has a curb-to-curb width of 19.5 feet and a structure length of 287 feet, with a maximum span length of 70 feet. The bridge is on a tangent horizontal alignment and is essentially level. The approach from the east is slightly curved and the approach to the west is tangent. The bridge has a minimum vertical clearance of 12'-10" and the roadway curb-to-curb width of 19.5 feet is also the rail-to-rail width. The vertical clearance throughout the structure varies, with the 12'-10" point occurring at one location due to the presence of an electrical box. Along the northern (westbound) side of the bridge, the vertical clearance of the roadway to the truss (with exception to the 12'-10" location) varies from 13'-6" to 13'-10". The vertical clearance along the southern (eastbound) side of the bridge varies from 13'-9" to 14'-0", and varies between 14'-1" to 14'-2" along the centerline of the roadway.

The original bridge was built in 1884 and consisted of a three span Pratt Truss iron superstructure supported by stone piers and abutments founded upon timber mats on native soil. From the west to the east the bridge was comprised of a 145-foot long two span swing truss span and an easterly 142-foot long fixed truss span. Reportedly, Pier 2 sustained vessel collision damage and required repair. In 1951, a rehabilitation project constructed a steel pile bent support system at Pier 2. This pile bent system transferred the superstructure and upper pier loads of Pier 2 to rock bearing H-piles. The rehabilitation project also added a pressure grouted steel jacket encapsulation to Pier 1 (swing pier). The steel pile support system of Pier 2 was rehabilitated in 1979 with the addition of concrete encasement of the H-piles and new bracing.

In 1991 to 1993 the bridge superstructure was replaced and a new pier added, creating the current four span multi-girder and ornamental truss bridge. From west to east the first two spans of the bridge consist of a continuous, two span swing span, supported by the rehabilitated center pier, west abutment and Pier 2; and a two span continuous fixed span supported by Piers 2 and 3 and Abutment 2. The west abutment is comprised of the original soil or rock supported cut stone and received a new concrete cap in 1993. The Swing Pier (Pier 1) was reinforced with cored steel pipe piles bearing in cored rock. The new pipe pile system transferred all loads to the rock bearing pipe piles. The swing spans are comprised of

multi girders with steel plate deck, and bituminous concrete overlay. The swing spans support the original iron truss as a decorative façade. Spans 3 and 4 consist of multi-girder spans with an exodermic (reinforced concrete slab on top of an unfilled steel grid) deck. The original iron truss decorative façade is supported independently of the new superstructure upon its original bearings at Pier 2 and Abutment 2. Pier 2 remains as a pile supported pier and received a new concrete cap. A new pier (Pier 3) was added, comprised of a concrete encased steel H-pile bent with concrete cap. The east abutment is comprised of the original soil or rock supported cut stone and received a new concrete cap in 1993. There is a 4-foot wide timber sidewalk located on the north side of the bridge consisting of pressure-treated 2 by 6 planks supported by timber stringers and steel cantilever brackets, protected by metal hand rail.

Historic

The bridge was listed in the U.S. Department of the Interior, National Park Service, National Register of a Historic Places February 12, 1987, as a rare example of an early movable iron bridge. The bridge is the oldest surviving movable bridge in Connecticut and one of the oldest such bridges in the United States. The bridge was substantially altered from its original construction in the 1993 rehabilitation.

It was constructed in 1884, when the Town of Westport contracted with Union Bridge Company of Buffalo, New York. The original design of the bridge consisted of a fixed and moveable span, both of which are Pratt through-trusses comprised of wrought iron, pin connected elements. The abutments on each bank are comprised of granite blocks. The center pier was of similar construction originally, but has since been encased in cast iron plate, behind which is a fill of gravel and sand.

Following an extensive rehabilitation campaign in 1993, the live load of the bridge is now carried by multi-girder superstructures. The 1884 wrought iron Pratt through-trusses have been retained as character-defining elements, though they no longer serve their purpose in supporting the bridge. In 2007, the bridge was officially renamed the "William F. Cribari Memorial Bridge" by the State of Connecticut.

Use of federal funding or permitting will require review under Section 106 of the National Historic Preservation Act of 1966. If this review determines that alterations to the bridge constitute an "Adverse Effect", additional evaluation through the FHWA (DOT Act of 1966) Section 4(f) is required. Replacement, or any alteration of the bridge would constitute an "Adverse Effect" under Section 106 and require evaluation through the FHWA (DOT Act of 1966) Section 4(f).

Accident Data

Traffic accident data between 2010 and 2014 was analyzed on Route 136 between mileposts 8.63 – located at the intersection with Riverside Avenue – and 8.74 – the approximate end of the bridge. The data indicates a high rate of accidents on or near the Cribari Bridge; the intersection of Route 136 and Riverside Avenue; and the commercial driveway between the intersection and the bridge. The total number of accidents in the five-year analysis period for the section of roadway studied shows a total of 59 accidents. 31 of those are attributed to the intersection of Route 136 and Riverside Avenue; 16 are attributed to the bridge or its immediate approaches; and 12 are attributed to various maneuvers into or out of commercial driveways. See Appendix I for accident data.

Approximately 85 percent of the accidents occurred on dry pavement conditions, with the remainder on wet pavement due to rain (approximately 12 percent) or snow/ice (approximately 3 percent). Both the intersection of Route 136 and Riverside Avenue and Route 136 and the commercial driveway have street lights, so all those accidents occurred under either natural or street lighting. And, while the bridge itself is not lighted, only one accident attributed to the bridge occurred under other than daylight conditions.

Nine accidents resulted in injury with the remainder resulting in property damage only. No fatalities were reported.

Approximately 69 percent of the accidents on or near the bridge involve sideswipes. The remaining accidents involve mostly rear-ends. There may also be a high number of unreported accidents involving a single vehicle hitting a bridge member. This assumption is reasonably made based on the evidence of continual damage on the bridge's guiderails, trusses, and approach guiderails. However, for all the evidence of damage to the bridge, only one accident in the analysis involved a single vehicle and a fixed object. Furthermore, that accident involved an injury, which may have contributed to it being reported.

While human error can be attributed to most traffic accidents in general, roadway geometry and traffic patterns can increase the chances of an accident occurring. All other factors held equal (e.g. weather, driver experience, tangent roadway section, etc...), it is reasonable to correlate the substandard curb-to-curb width of the Cribari Bridge with the high number of accidents recorded on or near the bridge. That is because, for a tangent roadway section on an almost flat profile, the number of accidents (reported and unreported) is very high. The comparison was made with bridges of similar roadway classification; between 200 and 500 feet in length; with average daily traffic between 10,000 and 15,000 vehicles; and carrying one lane in each direction. The 3 other bridges studied registered 1, 2, and 5 accidents (an average of 2.67 accidents per bridge) between them in the 5-year period in comparison to the 16 accidents that can be attributed to the Cribari Bridge. The common factor is that the other bridges are all over 28 feet wide curb-to-curb, meeting or exceeding the standard for this type of bridge.

Analysis of the accidents attributed to the intersection of Route 136 and Riverside Avenue shows that almost one-half of those accidents involved turning maneuvers; approximately one-third involved rear-ends; and approximately one-fifth involved sideswipes. Although turning maneuvers and rear-ends are normal accident occurrences at busy intersections such as this one, sideswipes are not. The sideswipes at or near this intersection can be reasonably correlated to the narrowness of the two lanes approaching the intersection from the east (Bridge Street). Further investigation of the traffic signal cycles; intersection configuration; the storage capacity of the right-turning lane from Bridge Street to Riverside Avenue; and proximity of commercial driveways in all legs of the intersection is required in order to fully address the high number of accidents at this location.

Analysis of the accidents attributed to the intersection of Route 136 and a commercial drive indicates that over 80 percent involved turning maneuvers. The remainder were rear-ends and sideswipes. Further investigation is required to fully account for the reasoning behind these accidents. However, the proximity of the intersection to the west of the driveway and the bridge to the east are factors that may be influencing the accidents. This is especially true due to the long queues on Route 136 just east of the intersection, which may cause drivers taking a left onto Route 136 from the driveway to "force" their way into the queue.

FIELD OBSERVATIONS

Recent inspections of this bridge by the Department identified needs for rehabilitation. All bridges in Connecticut are inspected on a biennial cycle. The biennial inspection cycle provides a continuous record of bridge condition. Guidelines for interpreting defects and deterioration and assigning a numeric rating to the structural element are contained in the Department's Bridge Inspection Manual and in Appendix J - Definitions. The condition evaluation establishes the structural and functional condition of the bridge components including the extent of deterioration and other defects.

This report was developed based upon review of the Department's Inspection Reports, bridge records and a site inspection performed in February 2015 by Close, Jensen and Miller, PC. Original construction plans, rehabilitation plans, maintenance reports, and other actions were reviewed. The findings and recommendations are summarized below.

Deck

The deck is in satisfactory condition (Overall rating = 6). The bituminous concrete overlay with membrane waterproofing displays extensive rutting up to 2 inches in the wheel lines. There are also potholes up to 3 inches deep and isolated longitudinal cracks. There is a steel deck plate in spans 1 and 2 (swing spans) that functions as a top flange for the girders. The steel form pans exhibit areas of rust; some areas are completely rusted through. The deck underside in spans 3 and 4 shows areas of heavy rust and large perforations in the deck pans. The bridge curbs are steel channel sections that exhibit minor scrapes and light rust. The average curb reveal is about 7 inches and the east approach curb has settled about 2 inches. There are six scuppers in each span and the grates are typically 50% clogged, but the down pipes are clear. The steel-armored open deck joints at the west abutment and Pier 2 have minor scrapes and gouges in the steel armor, and minor bituminous raveling along the edges. The joint at the east abutment in the eastbound lane has exposed rebar at the north end, which may be hazardous to snow plows.

Superstructure

The superstructure is in fair condition (Overall rating = 5). There are swing span screw jack bearings at the west abutment and Pier 2. The elastomeric bearings under the trusses and under the girders in spans 3 and 4 have areas of light rust and peeling paint. The elastomeric bearings for both trusses at Pier 2 have anchor bolt spacers with heavy rust and several spacers are missing washers. Spans 1 and 2 (swing spans) have a common plate that is continuous across all girders and Pier 1. Girders exhibit isolated areas of peeling paint and light rust. The girder bottom flanges in span 2 have minor scrapes and girder 1 in span 3 has a rolling defect in the web. The floor beams have heavy rust and accumulation of debris on the bottom flanges. The diaphragms have areas of heavy rust along the bottom and there are 10-inch diameter holes cut in the solid end diaphragms. The interior of the pivot girder has heavy laminar rust with minor section loss. The exterior pivot girder at Pier 1 has isolated areas of peeling paint, light rust, and debris accumulation under the trusses.

The sidewalk stringers have minor checks and wear. The sidewalk stringer support members have peeling paint and moderate rust. The first sidewalk support east of Pier 2 has connection bolts missing with the remaining bolts loose. The sidewalk support beams in the swing span have flame cut flanges over the substructure units, and a few stringer attachment channels have laminar rust. There is minor impact damage to the bridge metal beam rail. The west rail has disconnected posts and rubs against the southwest approach rail when the bridge opens.

Trusses

The trusses were retained during the 1993 rehabilitation for ornamental purposes only and carry no vehicular live load; therefore, trusses are no longer considered fracture critical to the superstructure. However, they are independently fracture critical by design and their collapse poses a safety hazard to vehicles and pedestrians. The trusses are in critical condition with an overall rating of 2. The bottom chord bottom flange plates have several areas of section loss and isolated rusted through holes. There is extensive collision damage to the truss members along the roadway. Several of the truss diagonal and vertical members have bent flanges, bent webs and cracks. The transverse sway braces at the truss ends display locations of bent flanges and dented angle legs due to vehicle impact. The field welded connections are typically uneven, sloppy and not ground smooth, and have isolated areas of light rust.

Mechanical & Electrical Systems

The most recent in-depth inspection report dated December 9, 2014 rated the Mechanical System Good (Overall rating = 7) and the Electrical System Good (Overall rating = 7). The machinery and electrical systems were repaired in 2012 due to previous storm damage.

Repairs are needed for the following deficiencies:

- Covers to the screw jacks need to be installed and oil leaks fixed.
- Screw jack limit switches should be replaced.
- The roller track of the pivot pier should be adjusted to level and the limit switch replaced.
- Add a maintenance platform at Pier 2.
- The power feed to the span drive motors should be relocated off of the truss.

Substructure

The substructure is in fair condition (Overall rating = 5). The stone masonry abutment stems have missing and deteriorated mortar at joints, voids up to 4-inches deep and isolated cracks. The west abutment concrete bridge seat has hairline cracks with efflorescence and areas of map cracking. The dry set stone masonry retaining walls along the channel have missing stones as large as 2.5 feet. The pedestals contain hollow areas, scaling and map cracking. The concrete pedestal under the north truss has light scale and the north face has map cracking and rust stains that had been coated with grout and re-cracked. The pier caps have areas of cracking with efflorescence and rust. The cap beam at Pier 2 has isolated areas of light rust and the carrier beams have light rust and section loss. The piles for Piers 2 and 3 are concrete encased H-piles with fiberglass jackets. At Pier 2, five of the six piles have loose or missing fiberglass jackets, and the exposed concrete has scaled with exposed reinforcement. The steel jacket at Pier 1 has heavy laminar rust and rusted through holes that expose the concrete. The exposed concrete has severe scales and spalls with exposed reinforcement. The footing at Pier 2 is exposed and undermined at both the southwest and northwest corners. The steel diagonal cross-bracing members at Pier 2 have heavy rust and up to 100-percent section loss. Due to its severe deterioration, the cross bracing at Pier 2 is rated critical (rating = 2) although the pier is rated to be fair overall. The southeast cross brace horizontal member is completely detached. There is scour up to 1.3-feet deep along the west abutment and up to 0.7-feet deep along Pier 1.

Hydraulic Adequacy and Scour

All elevations referenced to NAVD88.

The drainage area for this structure is 89.2 square miles. The water surface at the bridge is tidally affected from the Long Island Sound. The FEMA Flood Insurance Rate map (FIRM) indicates the effects of wave setup on the 100-year frequency tidal flood terminate at the south side of the bridge with the total (stillwater plus effects of wave setup) 100-year water surface elevation of 13 feet. The FIRM shows the 100-year water surface elevation on the north of the bridge is 10 feet. The Flood Insurance Study

(FIS) indicates stillwater elevations (due to the effects of astronomic tide and storm surge) for the 10-, 50-, 100-, and 500-year tidal flood frequencies of 8.1 feet, 9.7 feet, 10.4 feet, and 11.8 feet respectively, in the vicinity of the bridge. A storm-tide sensor deployed by the USGS for Hurricanes Irene and Sandy measured a peak storm tide elevation of 8.9 feet (08/28/2011) and 10.2 feet (10/29/2012) respectively, for these storms.

The Tidal Flood Profiles for the New England Coastline prepared by the U.S. Army Corps of Engineers indicates mean high water, mean low water, and 1-year frequency tidal flood elevations of 3.3 feet, -3.7 feet, and 4.7 feet respectively.

The Route 136 centerline profile and low chord of the bridge elevations are approximately 11.9 feet and 8.4 feet respectively. The machinery and electrical system of the swing span are positioned between approximate elevations 6.0 feet and 8.4 feet. The machinery and electrical system are exposed to flood flows and have received damage from previous storm flooding. The waterway opening of the bridge may be sufficient to pass the riverine flood without the effect of storm surge; however, due to the relative elevations of the low chord, machinery, and electrical systems, the existing bridge is susceptible to damage and closure from tidal storms.

The astronomic tide and storm surge elevations reported above do not consider the effects of sea level rise (SLR) that may be anticipated over the service life of the structure. Based on projections of SLR, the existing structure, with the mechanical and electrical systems at the current elevations, may be subject to more frequent damage from future SLR over the service life of the structure.

The channel is in fair condition (Overall rating = 5) based on the latest underwater inspection (12/29/2014) which reported:

- *“As compared to the 2012 Inspections, channel bottom elevations along the upstream (north) fascia, the abutments, Pier 1, and Pier 3 have remained relatively unchanged (variations up to 1.9’) with areas of localized degradation up to 2.4’ high at Span 3 at the north fascia and at the northwest corner of Pier 2. No repair recommendations requested at this time.”*
- *“At Pier 2, the previously observed scour has exposed the footing up to full-height (1.5’ high). The footing is undermined at the northwest corner 1.8’ long x 1.3’ high x 4’ penetration (previously noted as 2.1’ penetration) with the timber cribbing exposed. The footing is undermined at the southwest corner 1.5’ long x 8” high x 2’ penetration (previously noted as 2.5’ long x 5” high x 4” penetration) with the timber cribbing exposed. Design and install scour countermeasures for Pier 2. (2 CY).”*
- *“There is riprap up to 3’ diameter along the East abutment, and riprap near the northeast corner of Pier 3.”* The condition rating of the riprap was entered as “7”.

The “Scour Criticality” rating (NBI Item 113) in the database for this structure is an “8”, indicating low risk. No formal documentation has been found for this rating; however, it is likely based on initial screening performed by the USGS for the Department, the assumption that all of the substructure elements are founded on piles driven to rock and flood history. Portions of the bridge were constructed in 1870 and the bridge has survived several significant flood events without any apparent scour related damage or concerns.

However, the last available underwater inspection has reported scour and undermining at Pier 2. In addition, it has come to light that the abutment footing type and bottom elevations (depth of embedment) are unknown; therefore, the susceptibility of the abutment footings to scour and/or degradation in the river channel is also unknown. Given these conditions, the Item 113 rating of the existing structure should be reconsidered, at a minimum, revising “low risk” to “scour susceptible” and

the depth of the existing abutment footings should be investigated as well as the condition/extent of any existing scour protection.

The fender system at Piers 1 and 2 is constructed of timber piles and horizontal wales and is in fair condition (Overall rating = 5). The piles and wales exhibit splits and cracks ¼" to ½" wide throughout. The hardware exhibits heavy corrosion particularly in the tidal zone. Wales at random locations exhibit minor abrasion damage (possible vessel scrapes). The fender system appears to be substandard with respect to current AASHTO Design Code requirements.

Approach Roadways

The roadway approaches are in good condition (Overall rating = 7). The metal beam approach guide rails have areas of impact damage. The east approach pavement has areas of map cracking and settlement along the east abutment deck joint. The rail system along the east approach wingwalls does not meet AASHTO or CTDOT design codes for crash protection (does not provide system stiffness transitions to the trusses).

Utilities

There is an electrical box located in the sidewalk on the north side of the structure. There are also decorative holiday lights attached to all truss members above the deck. Overhead utilities are located at both approaches of the bridge and one utility line reaches from the west approach to the north side of Pier 1 (swing pier).

Property

The State's right-of-way for Route 136 at the bridge site encompasses Bridge No. 01349 entirely.

LOAD RATING & STRUCTURAL ANALYSIS

Load rating is the determination of live load carrying capacity of an existing bridge using existing bridge plans supplemented by information gathered during the field inspection. Engineering judgment is required to incorporate the effect of defects and deterioration in the load rating analysis. The AASHTO Condition Evaluation Manual recognizes load ratings at two levels—the inventory rating and the operating rating. The inventory rating generally corresponds to the design level of stress, and results in a live load that can safely use the bridge for an indefinite period of time, while the operating rating describes the maximum permissible live load that should not be exceeded on the bridge. Load ratings are computed and updated as part of the Department’s Bridge Inspection Program. The existing bridge is not posted for live load restriction.

This structure is unique in several ways with respect to structural adequacy. The following conditions exist and dictate the means and methods of structural analysis of the bridge system:

- The original abutments of the bridge are utilized for support of the new superstructure and the ornamental truss.
- Pier 1 was retrofitted in 1993 with a drilled shaft support system to support the loads of the new superstructure and the ornamental truss.
- Pier 2 utilizes the steel frame system built in 1951 and rehabilitated in 1979, to support the loads of the new superstructure and the ornamental truss.
- Pier 3 was added to the bridge in 1993 and supports only the new superstructure loads (the ornamental truss bears upon Pier 2 and Abutment 2, spanning Pier 3).
- The new superstructure of the swing span (Spans 1 & 2) was designed to carry the truss loads through moment connection to the fascia girders at the panel points and dead load end bearing at the span ends.
- The new superstructure of the fixed spans (Spans 3 & 4) was designed to have the truss be self-supporting, with bearing connections at Pier 2 and Abutment 2.

To assess the load carrying capacity of the bridge it was modeled in MIDAS Civil, finite element analysis and rating software to determine stress levels. The rating analysis was also performed utilizing MIDAS Civil analysis and rating software (see Appendix E for results of the analysis). The truss span from Pier 2 to Abutment 2 was modeled as a separate structure since it provides no live load capacity and does not contribute load to the new superstructure. The load rating and structural capacity of the bridge are summarized below:

Superstructure Spans 1 & 2: The live load rating capacity of these spans for all legal loads is greater than 1.0. (A rating factor of 1.0 indicates that the member(s) safely carry the applied loads).

Superstructure Spans 3 & 4: The live load rating capacity of these spans for all legal loads is greater than 1.0. (A rating factor of 1.0 indicates that the member(s) safely carry the applied loads).

Independent Truss Span Abutment 1 to Pier 2: The truss was not live load rated as it does not carry live load. The truss was analyzed for dead and wind load and found to be overstressed. A wind speed in excess of 100 mph produces an overstressed condition of the truss main members while a wind speed of 90 mph produces an overstressed condition for the cross bracing. The design wind speed is 125 mph as prescribed by AASHTO.

Independent Truss Span Pier 2 to Abutment 2: The truss was not live load rated as it does not carry live load. The truss was analyzed for dead and wind load and found to be overstressed. The truss main members and cross bracing exhibit acceptable stress levels for the design wind load of 125 mph.

Abutments 1 & 2: The abutments were not live load rated because as-built drawings do not exist. The abutments do not exhibit signs of distress and are thus assumed to be capable of carrying current loads.

Piers 1 & 3: An analysis of these piers indicated that stress levels are acceptable for design loads.

Pier 2 Support System: The support system for the pier does not meet the current code criteria based on today's standards in its current deteriorated condition. In its present condition the support system cap cross beams are stressed to 32.9 ksi with HL-93 loading in flexure. The yield capacity for the member is $33 \times 0.95 = 31.35$ ksi. When load rated these members yield rating as low as 0.87 for the HL-93 Truck (AASHTO Design Vehicles). The support system piles also exhibit substandard Capacity to Demand Ratio of 0.89 for axial compressive loads and combined axial compressive and flexural resistance overstress of 50% under HL 93 Truck loading (AASHTO Design Vehicles).

Vessel Collision Analysis: The pile supported system protecting Pier 1 consists of timber piles protected by timber fenders. There are six layers of 4-in. by 12-in. fenders spaced at 15 inches on center. The pile diameter varies from 13 inches minimum at the butt to 7 inches minimum at the tip. All timber material used in the fender system consists of Southern Yellow Pine.

An analysis of the existing fender system was performed. The design vessel incorporated in the analysis has a dead weight (DWT) of 1,000 ton, an overall length of 200 feet, and a bow depth of 27.2 feet as specified in the *Guide Specifications* Table 3.5.2-1, and with an impact velocity of 5.0 knots (approximately 8.44 feet/second). Based on the calculations contained herein, the fenders and piles would both fail in bending due to the design vessel collision.

According to the *Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges*, because of their relatively low cost, timber fenders are frequently used for protecting piers from minor vessel impact forces. It also states, however, that for the large collision impact loads associated with the design vessels in the *Guide Specifications*, the resulting timber fenders would have to be much larger and might be uneconomical in most circumstances (See G.S. Section 7.3.1.1).

Current practice recommends that a concrete pile system protected by steel or concrete fenders be used, as these materials have higher modulus and allowable stresses.

ANALYSIS & ALTERNATE EVALUATIONS

Preliminary alternates were assessed based on criteria including, but not limited to the State legislated criteria: (1) the functional classification of the highway; (2) the load capacity and geometric constraints of the bridge within its existing footprint and the availability of alternative routes; (3) the comparative long-term costs, risks and benefits of rehabilitation and new construction; (4) the requirements of state standards for geometric design; (5) disruption to homes and businesses; (6) environmental impacts; (7) the potential effects on the local and state economies; (8) cost-effectiveness; (9) mobility; (10) safety, as determined by factors such as accident history for motorists, pedestrians and bicyclists; and (11) the impact on the historic, scenic and aesthetic values of the municipality in which the bridge is or may be located. The determination of “No Adverse Effects” or “Adverse Effects” would be reviewed and determined during the Section 106 review process. During this review, the baseline integrity of the structure will be analyzed, focusing on aspects including but not limited to the number of members being replaced for each alternate, methods of replacing or maintaining truss members, parameters of any new members and their ability to visually match those being replaced, as well as documentation of members that have already undergone replacement/rehabilitation. Alternates that show promise or feasibility within the parameters were pursued.

The conditions of the structure which affect public safety and functionality include:

- The substandard load capacity of the bridge.
- The substandard roadway barrier system.
- The substandard roadway functional width and vertical clearance.
- The lack of a solid roadway barrier system during bridge openings.
- The substandard wind load capacity of the trusses (fracture critical).
- The substandard east approach wingwall traffic barrier.
- The hydraulic inadequacy and susceptibility of the bridge to flooding (10-year frequency event).
- The substandard fender system.

The options for rehabilitation are limited by several constraints:

- The east approach embankment and stone wingwalls preclude substantial widening.
- The ornamental truss configuration limits access for construction.
- The load carrying capacity of the substructure limits the addition of load.
- Access to the bridge is difficult due to the tidal fluctuation and sensitive mud flats it crosses.

Three alternates were investigated and deemed not viable as discussed below.

No Action - There are several bridge conditions which pose public safety concern and cannot be ignored. These include but are not limited to the substandard live load capacity of the Pier 2 support system; the substandard bridge railing system; the capacity of the trusses to withstand wind loading; and the damaged truss condition. If left uncorrected, these conditions will continue to worsen and eventually create a hazardous condition for anyone using the bridge. Continued and prolonged use of the structure without any form of repair or rehabilitation would eventually result in a failure of one or more of these items, resulting in closure of the bridge in addition to the obvious safety hazards. One of the goals of the Department of Transportation is to maintain the State’s Highway and Bridge systems in a safe, efficient manner balanced with the needs of the travelling public; therefore, not performing any corrective measures is not a feasible option.

Minor Repairs – This rehabilitation alternate would only involve repair of the damaged and deteriorated elements: repairs to the truss system and Pier 2 with additional minor repairs to the structure throughout. However, this option would not correct the cause of the damage that occurs to the structure: i.e. the existing railing would be maintained, resulting in vehicles continuing to strike and damage the truss members; detrimental to both the structure and motorists. Failure to incorporate such preventative measures would increase the frequency of rehabilitation projects that would be required, and is therefore not realistic economically. The goal of a rehabilitation project should be to get in, get out, and stay out, as opposed to the need for recurrent minor repairs. The minor repair option also does not result in any improvements in safety to the travelling public along the bridge.

One-way Bridge Access – A rehabilitation alternate which would replace the existing bridge railing system with a safety and crash compliant system and provide clearance to the trusses, in addition to performing repairs to the damaged and deteriorated elements, was investigated. The installation of a new rail system would reduce the existing bridge roadway width, making the bridge unsuitable to provide 2-way traffic; and therefore requiring a one-way lane configuration over the bridge (westbound). Reducing the bridge to one lane would allow for wider shoulders on each side, which would also accommodate bicyclists. The option would permanently re-route eastbound Route 136 traffic north along Riverside Avenue/Route 33 to Route 1. As noted in the Appendix G traffic analysis, conditions along Route 1 are not ideal, and the permanent redirection of such a substantial amount of traffic would only worsen the already inadequate traffic operation. The implementation of a one-way lane would also restrict emergency medical services, law enforcement, and the fire department from efficiently accessing key locations within Westport. Implementation of a Contra-flow bike lane (a lane in which traffic flows in the opposite direction of the surrounding lanes) is not an ideal situation, as they encourage cyclists to ride against traffic, which is contrary to the rules of the road and is a leading cause of bicycle/motorist accidents. Public outreach with the Town of Westport, as well as an internal review of this option by CTDOT Traffic and Intermodal Planning, had both generated responses that implementation of a one-way roadway is not an acceptable solution as a rehabilitation alternate.

After careful consideration two alternates were selected for further consideration; Alternate A: Major Rehabilitation and Alternate B: Structure Replacement.

Life Cycle Cost Analysis - A life-cycle cost analysis has been prepared to estimate the anticipated present value for both alternates reviewed in this Rehabilitation Study Report. An analysis period of 75 years is shown for the two alternates in Appendix F. The life cycle cost is \$41.27 million for Alternate A: Major Rehabilitation and \$41.43 million for Alternate B: Structure Replacement. The scenarios applied to the alternates are as follows:

Alternate A: Major Rehabilitation

- A 25-year service life is assumed for the rehabilitation work, which is fairly consistent with the history of rehabilitation projects performed on the structure (previous construction being required in 1951, 1979 and 1993).
- At the end of the 25 year life of the rehabilitation project, a new rehabilitation project is assumed to take place, which will primarily involve repairs to the deck and superstructure with repairs-as-needed to the substructure.
- At year 40, it is assumed the structure will be in such a condition where full replacement would be required. The future value is consistent with the cost for the complete replacement alternate.
- At year 65, consistent with the history of projects for the structure, a minor rehabilitation project is assumed to take place. The rehabilitation project will include minor repairs to the structure and full painting of the superstructure (life cycle of painting is typically 25 years).

- Milling and paving is assumed to take place every 15 years, up to year 40, where a full structure replacement is assumed, and is assumed to continue every 15 years after the full replacement.
- Due to the mechanical and electrical systems being located below the 10-year storm event, an additional cost is shown every 10 years for its repair/replacement as a result of storm damage. The future value for this item is consistent with the cost for the electrical/mechanical repair that had taken place in 2012. For consistency with the Alternate B analysis, once the full replacement of the structure is performed (year 40), the location of the mechanical/electrical equipment will be at an elevation outside of the 500-year storm event. It is assumed that any additional repairs or rehabilitation would be performed during the major/minor rehabilitation projects shown in the analysis.

Alternate B: Complete Structure Replacement

- A 75-year service life is assumed for the complete replacement of the bridge.
- At year 25, consistent with Alternate A after full structure replacement, a minor rehabilitation project is assumed to take place. The rehabilitation project will include minor repairs to the structure and full painting of the superstructure (life cycle of painting is typically 25 years).
- At year 50, a rehabilitation project is assumed to be required for full structure painting (based upon previously noted life for paint) with some additional repairs to the substructure.
- Milling and paving is assumed to take place every 15 years.
- As noted in the bullet item for Alternate A regarding mechanical/electrical maintenance, with a new structure in place, the mechanical/electrical equipment will be installed above of the 500-year storm event; and it is assumed that any additional repairs or rehabilitation would be performed during the major/minor rehabilitation projects shown in the analysis.

Traffic Analysis - A traffic analysis of the surrounding intersections was prepared to determine the feasibility of closing the bridge and detouring traffic during certain construction tasks. The detour route would direct traffic along Route 33 (Riverside Avenue), Route 1 (Post Road to State Street), and Route 136 (South Compo Road) in both directions. A detailed report can be found in Appendix G.

As a result of the traffic analysis, it was determined the noted detour route would not operate at an acceptable level of service during extended construction periods without the need for significant roadway capacity and intersection improvements. The use of a temporary bridge or the existing bridge to direct traffic during construction in lieu of a detour would therefore be implemented for both options presented in this report.

Bicycle/Pedestrian Access – Bridge No. 01349 serves as a vital bridge that connects communities, business developments, and the train station for Westport south of I-95. It is therefore understood that consideration must be given to pedestrians and cyclists for the options presented below. Route 136, within the vicinity of the project, is part of the East Coast Greenway On-Road and interest is shown in Westport for more and better facilities for bicycles and pedestrians, as outlined in Westport’s 2007 Town Plan for Conservation and Development. The Town’s goal, according to the Westport Town Plan, includes preserving open space and creating greenways, improving connectivity of sidewalks and bicycle paths along and near roads, and providing alternatives to automobiles, including pedestrian, bicycle, and transit. The Westport Town Plan goes on to recommend improvements for pedestrians, such as encouragement of sidewalks in higher density areas, sidewalks be considered along arterial and collector roads, and that the existing sidewalk system be maintained and improved. Recommendations for Bicycles per the Town Plan include marking and maintenance of the existing and new bike lanes, promoting of bicycle use, and the establishment of new safe bicycle routes along arterial and collector

roads. The inclusion of such improvements for accommodation of cyclists and pedestrians have been reviewed and incorporated, where applicable, as a part of the alternates described below.

Alternate A: Major Rehabilitation

This alternate consists of remediating only the bridge's major public safety and functional issues. The major items of work would include:

- Widening the ornamental truss and installing a new barrier system.
- Reconstruction of the Pier 2 support system.
- Increase the vertical clearance to 14'-3" to satisfy functional adequacy.
- Adding a moment connection to the trusses at panel points to resist wind load stresses.
- Deck patching, membrane waterproofing and paving.
- Complete painting of the superstructure steel and ornamental truss.
- Installing a solid roadway barrier system at both approaches (precludes vehicles from accidentally driving into the waterway).
- Structural steel repairs.
- Substructure patching.
- Replacement of the existing fender system.

The following minor repairs would be included:

- Install screw jack covers and correct oil leaks.
- Replace screw jack limit switches.
- Adjust the roller track of the pivot pier level and replace the limit switch.
- Add a maintenance platform at Pier 2.
- Relocate the power feed to the span drive motors off of the truss.

Widening the ornamental truss and installing a new barrier system would require separating the leaves and moving them outward approximately 2 feet in each direction. Separating the truss leaves to accommodate the new barrier systems would require widening the top chord bracing and portals and extending the bottom chord bracing system. Widening the truss of the movable span will require the screw jack motors and machinery and limit switches to be modified. It will also require modification to the pivot pier limit switches. Widening the truss will offset the top bracing connection plates thus increasing the vertical clearance to 14'-1", which satisfies the criteria for functional adequacy.

To achieve the design standard for vertical clearance over a Minor Urban Arterial Route (Route 136), a height of 14'-3" would be required. The connection points of the truss system currently control the vertical clearance along the bridge; however, the vertical clearance along the centerline of the roadway is consistently 14'-1" or greater. Once the truss is splayed, the connection points of the truss would be past the proposed guiderail and outside of the traffic envelope, resulting in 14'-1" as the new minimum vertical clearance. A functional vertical clearance could then be achieved by raising the truss 2 inches. Construction of a new barrier system would also require the construction of a new fascia beam support system. The existing sidewalk would be shifted to the north to accommodate the widening of the truss, requiring lengthening the cantilever bracket supports and reconstructing the sidewalk and rail system. To minimize the load increase to the existing substructure, it is not proposed to increase the sidewalk width. Temporary support of the leaves or removing the existing truss to an off-site location would be required to facilitate the widening. Conceptual details of the proposed truss and superstructure widening and proposed barrier system and attachment are included in Appendix H. During the support or temporary removal and replacement operations, the bridge would be closed to vehicular and pedestrian traffic (See Appendix H - Conceptual Schedule).

Replacement of Pier 2 was investigated and the concept dismissed. Replacement would require removing and replacing the superstructure of spans 3 and 4 in their entirety, which would be cost prohibitive. Another concept that was investigated and dismissed was coring through the existing stone column of Pier 2 and constructing reinforced concrete shafts. The spatial constraints of the beams and end diaphragms would also require removal and replacement of the spans 3 and 4 superstructure.

Reconstructing the existing steel bent support system was pursued. Reconstruction of the Pier 2 support system would require removal and an upgraded replacement of the existing system. The new support system would be constructed of concrete encased galvanized steel beams and bracing with cathodic protection. This work would require partial removal of the deck for pile removal and driving or augering operations. This work would also require the closure of the bridge to vehicular and pedestrian traffic (See Appendix H).

Deck patching, membrane waterproofing and paving operations would require the closure of the bridge to vehicular traffic. Painting of the bridge will require a negative air containment system due to the presence of lead paint. This work would also require the closure of the bridge to vehicular and pedestrian traffic due to intrusion of the containment enclosures and the blast cleaning equipment. Installing a solid roadway barrier system would require the closure of the bridge to vehicular traffic.

Structural steel repairs will be performed and the stone masonry substructure will be repointed as needed and the reinforced concrete substructure repaired as necessary.

The final roadway width will be maintained at 19.5 feet curb to curb. It is anticipated that the roadway would be closed to vehicular and pedestrian traffic for the duration of construction. A temporary bridge would be constructed to maintain traffic during the timeframe the existing bridge will be closed. The complete rehabilitation construction duration is estimated to be approximately 2.5 to 3 years, which includes two winter shut down periods. During this time, restrictions will also be in place for aquatic vessels, which will not be allowed to cross beneath the bridge during certain tasks being performed beneath the existing structure. Improvements would also be performed at the intersection of Route 136 (Bridge Street) and Riverside Avenue. Improvements would include modification of the traffic signal to provide added green time for the Bridge Street movement phase in the AM and increasing the length of the Bridge Street westbound right-turn storage lane to approximately 105 feet. As noted in Appendix G, the proposed modifications would bring the intersection to an acceptable operating level, improving the flow of traffic along Route 136, and will likely result in a reduction of the number of rear-end accidents that occur along the bridge.

Details associated with the rehabilitation are included in Appendix H.

ADVANTAGES

- ✓ Addresses structural deficiencies
- ✓ Maintains historic elements
- ✓ Less initial cost than full replacement option



DISADVANTAGES

- Bridge remains functionally obsolete
- The bridge remains hydraulically inadequate and the machinery susceptible to storm flooding (10+ year storm frequency)
- Substandard pedestrian and bikeway facilities remain
- Does not address the substandard west approach wingwall traffic barrier
- Causes an adverse effect to the historic elements

Alternate B: Structure Replacement (New Pratt Truss Swing Span)

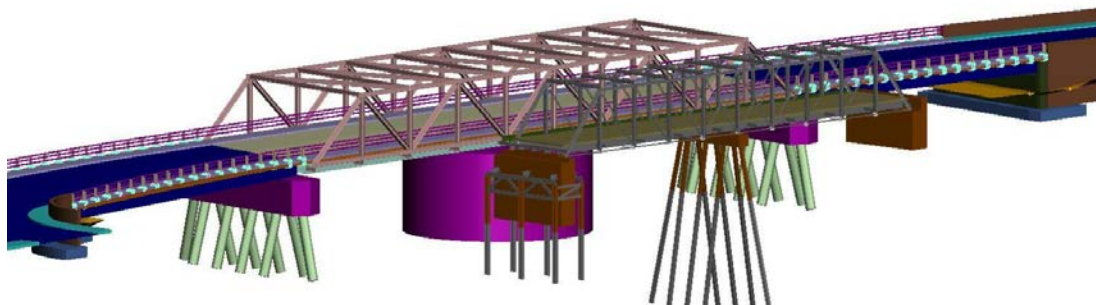
This alternate consists of replacing the existing structure with a new four span structure, consisting of two multi-girder fixed spans and a two span Pratt Truss swing span (see Conceptual plans). The new bridge would be constructed to the north of the existing bridge and utilize the existing bridge to maintain traffic during construction. The approach roadways would be reconstructed and realigned to accommodate the new structure.

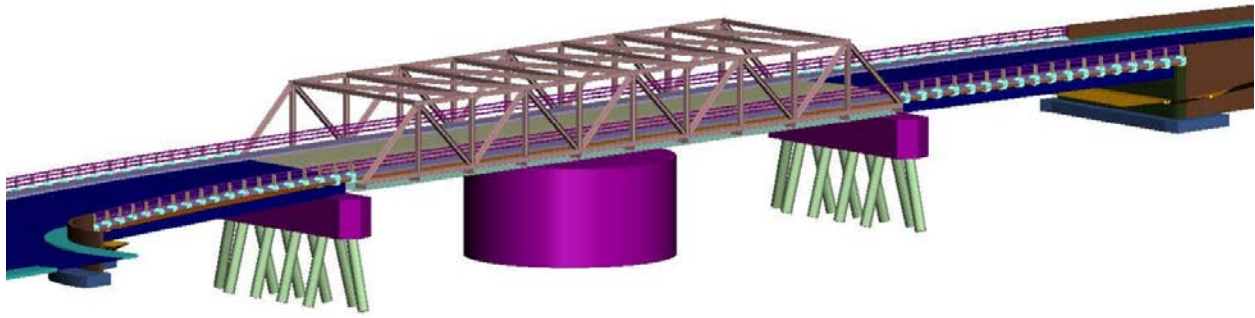
The swing span would be located at the center of the structure and cantilevered at a length of 105 feet each direction. The approach spans would be approximately 100 feet in length each (see Sketch plans). The west abutment would be constructed behind the existing stone walls that line the channel. In-line retaining walls would be used to contain the east approach roadway. Abutments and piers would be pile supported. The superstructure would utilize a reinforced concrete deck and metal bridge rail. The bridge would have a roadway width of 47 feet to accommodate an 11-foot travel lane, a 5-foot shoulder/bikeway with a 1.5-foot marked buffers between the vehicular travel lanes and bike lanes, and a 6-foot sidewalk in each direction. The structure would be skewed at an angle of 10-degrees to accommodate the channel alignment. The bridge's low chord and mechanical and electrical systems would be constructed above the 500-year storm elevation. The bridge underclearance would be increased and the approach roadways raised. New signals and gate arms and solid barriers would be installed at each approach roadway.

Similar improvements to those noted in Alternate A would be applied to the intersection of Route 136 (Bridge Street) and Riverside Avenue. The existing signal timing would be modified to provide added green time for Bridge Street during the AM and the Bridge Street westbound right-turn storage lane would be lengthened to approximately 105 feet. An additional lane across the bridge to replace the existing right-turn lane was looked into, as noted in Appendix G; however, only a slight improvement to the operation of the intersection was observed, and a single lane in each direction across the bridge is recommended. The intersection modifications in addition to the new structure width would not only reduce the number of rear-end collisions, but also the sideswipes that commonly occur along the existing bridge.

Construction of the proposed off-alignment replacement bridge is anticipated to take approximately 2.5 to 3 years, including two winter shutdown periods, and vehicular and pedestrian traffic would be maintained throughout construction on the existing bridge.

The structure proposed is for comparative purposes only. Should a bridge replacement option be selected, a structure-type study would determine the actual bridge type to be selected and the geometry and lane capacity. For relative location comparison, the structure modeled for this report is depicted below, first with the existing fixed spans shown, followed by a depiction of just the replacement bridge.





ADVANTAGES

- ✓ Longest structure life
- ✓ Addresses all functional, structural, and public safety issues
- ✓ The bridge would be hydraulically adequate
- ✓ Adds adequate and safer bicycle and pedestrian access



DISADVANTAGES

- Highest initial cost
- Loss of historic truss

COST CONSIDERATIONS

Appendix B contains an itemized cost estimate for each alternate. The quantities reflected in each estimate represent roadway and structure items. The anticipated total costs for Alternate A and B are:

Alternate A: Rehabilitation	\$ 19,800,000
Alternate A: Rehabilitation Life Cycle Cost	\$ 41,269,335
Alternate B: Structure Replacement (New Pratt Truss Swing Span)	\$ 35,800,000
Alternate B: Structure Replacement Life Cycle Cost	\$ 41,431,486

RECOMMENDATIONS

The No Action alternative does not resolve any of the issues. The Minor Repairs alternate retains the historic character but does not resolve the continued structural damage that occurs to the truss. The One Way Bridge alternate resolves the ongoing bridge damage and retains the historic character, but causes un-resolvable traffic problems for vehicles and bicycles.

The two options that go a long way to address the purpose and need are the Major Rehabilitation alternate and the Structure Replacement alternate. The lifecycle costs for both options are essentially the same and, as such, it is recommended that further in-depth studies of various bridge replacement alternatives be completed before any determinations can be made whether to rehabilitate the bridge or to replace it. A more in-depth benefit/cost analysis is also warranted to take into consideration the cost of lost commuter time due to traffic queues as well as the environmental impacts associated with fuel emissions resulting from idling vehicles. It should also be noted, that in addition to the high initial cost of the major rehabilitation, a number of significant functional deficiencies would still remain unaddressed with a major rehabilitation. These include the geometric deficiencies of the existing bridge for handling the volume of traffic, continued susceptibility of damage to mechanical and electrical features of the structure due to 10-year or greater storm events, and the notable fact that Route 136 is part of the East Coast Greenway, and the substandard width is not acceptable for safe passage for future bikeway and pedestrian needs. Further engagement of all affected stakeholders would be initiated in a public outreach process for developing and assessing bridge replacement alternatives before any determination is made for bridge rehabilitation or replacement.

APPENDICES

Appendix A	Photographs
Appendix B	Cost Estimates
Appendix C	Department Inspection Report
Appendix D	FEMA FIRMette
Appendix E	Load Rating & Structural Analysis
Appendix F	Life Cycle Cost Analysis
Appendix G	Traffic Analysis
Appendix H	Alternate A: Major Rehabilitation Details
Appendix I	Accident Data
Appendix J	Definitions