CAPE COD CANAL HIGHWAY BRIDGES
BOURNE AND SANDWICH, MASSACHUSETTS
MAJOR REHABILITATION EVALUATION REPORT

APPENDIX A
ENGINEERING RELIABILITY ANALYSIS

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APPENDIX A

ENGINEERING RELIABILITY ANALYSIS

1. PURPOSE

This appendix contains an engineering analysis which demonstrates the criticality and reliability of key elements of the Bourne and Sagamore bridges. The results of this analysis form the basis for the economic evaluation of the base condition versus alternative schemes for repair or replacement.

2. PROJECT OVERVIEW

a. Description – Bourne Bridge

The Bourne Bridge is one of three Cape Cod Canal crossings and carries vehicular and pedestrian traffic on State Route 28 across the Cape Cod Canal, Sandwich Road and the Massachusetts Coastal Railroad. There are two vehicular bridges, the Bourne and Sagamore, and one railroad bridge. Figures A-1 through A-11 exhibit the various features and bridge components discussed in this appendix.

The Bourne Bridge, constructed in 1933 under the direction of the United States Army Corps of Engineers, is a seven span bridge with four approach spans and three main spans. Spans are labeled 7, 5, 3, 1, 2, 4 and 6 from south to north with the main center span designated span 1, the main side spans designated span 3 to the south and span 2 to the north, and the approach spans alternating with even number designations to the north and odd number designations to the south. Piers are labeled 5, 3, 1, 2, 4 and 6 from south to north with piers 1 and 2 located at either shore of the channel.

The total length of the bridge is 2,684 feet. The main spans of the Bourne Bridge, spans 3, 1 and 2, are composed of steel trusses forming three continuous spans. Span 1 extends over the Cape Cod Canal, is 616 feet in length, and provides a vertical clearance of 135 feet above mean high water. Span 1 is a through arch truss suspended span with 22 galvanized strand suspender cables, 11 on each truss, to suspend the roadway deck and floor system. The two side spans, each 396 feet in length, are deck trusses that transition over piers 1 and 2 into span 1.

The four approach spans are simply supported deck trusses ranging in length from 208 feet to 270 feet with spans 4 and 6 to the north of the canal and spans 5 and 7 to the south of the canal. The bridge approaches consist of a 150 foot long multi-chamber abutment at each end.

The roadway is composed of four 10'-0" wide lanes with a 6'-8" wide sidewalk along the west fascia and a 2'-0" brush curb along the east fascia.

The overall configuration of the Bourne Bridge means it a “fracture critical” bridge. It has non-redundant steel tension members, primarily the trusses, which defines this bridge as fracture critical.

i) Superstructure

The approach span trusses (spans 7, 5, 4 and 6) and main side span trusses (spans 3 and 2) vary in depth from 22 feet to 45 feet, as measured from the centerline of the bottom chord to the centerline of the top chord. The main center span arch truss (span 1) varies in depth from 50 feet to 93 feet.

The approach span trusses have eight panels each with truss joints labeled 0 to 8 from south to north. Truss panels are 30'-0" long in span 7; 30'-9" long in span 5; 30'-0" long in span 4; and 26'-0" long in span 6.

The main side span trusses have nine panels each and the main center span arch truss has 14 panels with all panels 44'-0" long. Truss joints in these three spans are labeled symmetrically about the midspan of span 1 at joint 16 with the joints to the north differentiated with a "prime" designation. From south to north, these truss joints are designated 0 to 9 in span 3; 9 to 16, then 15' to 9' in span 1; and 9' to 0' from south to north in span 2.

The truss floor system is composed of sixty-nine 5'-0" deep floorbeams located at each truss joint in each truss span. Nine stringers and three support channels span between the floorbeams and support the roadway deck, sidewalk and brush curb. The stringers are spaced 5'-0" on center. The floorbeam ends are connected at the trusses, with the exception of the 11 center floorbeams in span 1 which are supported at each end by a pair of galvanized strand suspender cables for a total of 44 individual suspender cables.

ii) Abutments

The abutments are hollow cell concrete structures each composed of three chambers. Within the abutments are concrete bents or transverse chamber walls forming bays ranging in length from 28'-1" to 30'-6". The bents and chamber walls support six reinforced concrete T-beams spaced 7'-0" on center.

iii) Piers

The two channel piers, piers 1 and 2, each consist of two columns that share a common 25'-0" deep footing with the columns set on individual pedestals. The top of the pedestals for each column, above the top of the footing, is 34'-0" and 27'-0" for piers 1 and 2, respectively. The upper 14'-0" of the pedestal is clad with a granite stone facing with a depth of stone into the footing of 2 feet to 3 feet. The hollow concrete columns are 24'-0" by 24'-0" at the base and 15'-0" by 15'-0" at the bearing level and are joined at the top by a tapered strut that has a minimum depth of 23'-0" located at the midspan of the strut. The distance from the top of the columns to the top of footings is 111'-0" and 104'-0" for piers 1 and 2, respectively.

Piers 5, 3, 4, and 6 consist of two solid concrete columns that share a common 13'-0" deep footing. Only pier 6 has columns founded on individual pedestals while the remaining piers have the columns founded directly onto the footing. For pier 6, the top of each pedestal is 7'-0" above the top of the footing. The solid concrete columns are 20'-0" by 20'-0" at the base and 14'-0" by 14'-0" at the bearing level at piers 3 and 4. For piers 5 and 6, the solid concrete columns are 18'-0" by 18'-0" at the base and 12'-0" by 12'-0" at the bearing level. All four piers have their columns joined at the top by a tapered strut that has a minimum depth of 16'-0" located at the midspan of the strut. The distance from the top of the columns to the top of the footings is 54'-6" for pier 5, 60'-0" for piers 3 and 4 and 63'-0" for pier 6.

iv) Deck

The deck on the truss spans is a steel grid deck filled with 5" of lightweight concrete. The wearing surface is 2" thick bituminous concrete (Rosphalt). The underside of the deck is hidden by stay-in-place forms. The deck within the abutment spans is 9" thick reinforced concrete.

b. Maintenance History – Bourne Bridge

Table 1 below summarizes the maintenance and repair history of the Bourne Bridge.

Table A-1 Bourne Bridge Maintenance and Repair History	
YEAR	**WORK PERFORMED**
1938	Painted superstructure.
1938	Sealed coated wearing surface - sheet asphalt.
1947	Painted superstructure.
1949	Replaced bituminous pavement.
1952	Painted superstructure.
1958	Painted superstructure.
1959	Replaced 4 anchor bolts (Piers 3 and 5).
1963	Resurfaced roadway and sidewalk; new curbing; new scuppers; replaced 5’ strip of deck concrete adjacent to the sidewalk and the brush curb; electrical work; concrete repairs; access ladders; platforms and downspouts.
1967	Painted superstructure.
1969	Pressure grouting of cracks in concrete abutments and piers.
1971	Painted railings.
1973	Painted superstructure.
1976	Repaired two stringers, Span 4; replaced sidewalk bracket, Span 1, removed bird droppings from abutments; removed two pairs of hanger cables for testing and replaced with new cables.
1979	Removed existing deck and replaced with lightweight concrete filled steel grid deck; installed new waterproofing membrane and bituminous wearing surface; strengthened upper and lower bracing in Spans 4 to 7; repaired over 250 members; repaired or replaced over 200 gusset/stay plates; replaced approximately 3000 deteriorated rivets with high strength bolts; installed new roadway joints; and painted superstructure.
1984	Placed new waterproofing membrane on sidewalk and curb.

Table A-1 Cont. - Bourne Bridge Maintenance and Repair History	
YEAR	**WORK PERFORMED**
1986	New hanger cables installed; new drainage pipes installed; new waterproofing on curb; patched spalls and injected cracks on abutments, piers, and parapets; electrical work; painted superstructure.
1988	Removed existing bituminous waterproofing membrane and top 1-1/2 inch of deck concrete on abutments; placed new 1-1/2 inch micro-silica overlay; new waterproofing membrane and bituminous concrete wearing surface.
1992	Painted superstructure.
1997	Repaired/replaced deck joints at South Abutment, Pier 3 and North Abutment.
1999	Replaced deck joint at Pier 4; major concrete repairs to abutments and piers.
2000	Replaced concrete parapets; repaired sidewalk and curbs; replaced waterproofing membrane and bituminous wearing surface on deck and abutments; miscellaneous electrical work.
2001	Major substructure rehabilitation including: concrete spall repairs to piers, abutment seats, abutment chamber walls and bents and concrete stringer repairs within chambers.
2004	Painted superstructure with work completed in 2006.
2010	Deck rehabilitation contract performed. Removed the existing asphalt pavement and waterproofing membrane on both abutments and the steel superstructure deck; repaired concrete substrate on abutments; repaved entire length of bridge with Rosphalt.
2012	Steel repairs throughout the entire length of the bridge including gusset plate patch plates, replacement of sway bracing, replacement of missing rivets with bolts at member connections and lacing bar connection, removal of fatigue sensitive weld details on truss members, floorbeams and stringers and replacement of deck drainage support brackets with new drainage downspouts. \$6.8 million (combined with Sagamore Bridge Steel Repairs – Total \$9.7 million).

c. Description – Sagamore Bridge

The Sagamore Bridge is one of three Cape Cod Canal crossings and carries vehicular and pedestrian traffic on State Route 6 across the Cape Cod Canal, Sandwich Road and the Massachusetts Coastal Railroad. The Sagamore Bridge, completed in 1935 under the direction of the United States Army Corps of Engineers, is a three span bridge. Spans are labeled 3, 1 and 2 from south to north with the main span designated span 1, and the side spans designated span 3 to the south and span 2 to the north. Piers are labeled 1 and 2 from south to north with piers 1 and 2 located at either side of the canal.

The total length of the bridge including the abutment spans is 1,833 feet. The main spans of the Sagamore Bridge, spans 3, 1 and 2, are composed of steel trusses forming three continuous spans for a total length of 1,408 feet. Span 1 extends over the Cape Cod Canal, is 616 feet in length, and provides a vertical clearance of 135 feet above the navigation channel at mean high water. Span 1 is a through arch truss suspended span with 22 galvanized strand suspender cables, 11 on each truss, to suspend the roadway deck and floor system. The two side spans, each 396 feet in length, are deck trusses that transition over piers 1 and 2 into span 1.

The bridge approaches consist of a 225 foot long reinforced concrete multi-chamber abutment at the south end and a 200 foot long reinforced concrete multi-chamber abutment at the north end.

The roadway is composed of four 10'-0" wide lanes, two in each direction with a 6'-8" wide sidewalk along the east fascia and a 2'-0" brush curb along the west fascia.

A system of ladders, platforms and catwalks provides inspection and maintenance access from inside of each abutment to the bridge seats, the floor system throughout the full length of the bridge and the pier caps. A separate system of ladders, platforms and catwalks provides access along the east truss lower chord above the roadway from truss joint L11' to L16, the east truss lower chord to the upper chord at truss joint 16 and the east truss upper chord to the west truss upper chord at U16.

The overall configuration of the Sagamore Bridge means it a “fracture critical” bridge. It has non-redundant steel tension members, primarily the trusses, which defines this bridge as fracture critical.

i) Superstructure

The main side span trusses (spans 3 and 2) vary in depth from 44'-9" to 93'-0", as measured from the centerline of the bottom chord to the centerline of the top chord. The main center span arch truss (span 1) varies in depth from 50 feet to 93 feet.

The side span trusses have nine panels each and the center span arch truss has 14 panels with all panels 44'-0" long. Truss joints in these three spans are labeled symmetrically about midspan of span 1 at joint 16 with the joints to the north differentiated with a “prime” designation. From south to north, these truss joints are designated 0 to 9 in span 3; 9 to 16, then 15' to 9' in span 1; and 9' to 0' from south to north in span 2.

The truss floor system is composed of thirty-three 5'-0" deep floorbeams located at each truss joint in each truss span. Nine stringers and three support channels span between the floorbeams and support the roadway deck, sidewalk and brush curb. The stringers are spaced 5'-0" on center. The floorbeam ends are connected at the trusses, with the exception of the 11 center floorbeams in span 1 which are supported at each end by a pair of galvanized steel strand suspender cables for a total of 44 individual suspender cables.

ii) Abutments

The abutments are hollow cell concrete structures each composed of four chambers. Within the abutments are concrete bents or transverse chamber walls forming bays ranging in length from 26'-10" to 31'-4". The bents and chamber walls support six reinforced concrete T-beams spaced 7'-0" on center. Chamber 3 of the south abutment spans over Sandwich Road and has an additional concrete slab above Sandwich Road and below the bridge deck which is supported by two T-beams and acts as a floor for the interior of chamber 3.

iii) Piers

The two channel piers, Piers 1 and 2, consist of two columns that share a common 25'-0″ deep footing with the columns set on individual pedestals. The top of each pedestal is 37'-0″ above the top of footing. The upper 16'-0″ of the pedestals is clad with stacked granite stone facing with a depth of stone into the footing of 2 feet to 3 feet. The hollow concrete columns are 24'-0″ by 24'-0″ at the base and 15'-0″ by 15'-0″ at the bearing level and are joined by a tapered strut that has a minimum depth of 23'-0″ located at the midspan of the strut. The top of each column is 114'-6″ above the top of footing.

iv) Deck

The deck on the truss spans is a steel grid deck filled with 5" of lightweight concrete. The wearing surface is 2" thick bituminous concrete (Rosphalt). The underside of the deck is hidden by stay-in-place forms. The deck within the abutment spans is 9" thick reinforced concrete.

d. Maintenance History – Sagamore Bridge

Table 2 below summarizes the maintenance and repair history of the Sagamore Bridge.

Table A-2 - Sagamore Bridge Maintenance and Repair History	
YEAR	**WORK PERFORMED**
1938	Paint superstructure.
1938	Seal coat wearing surface - sheet asphalt.
1942	Paint railings.
1947	Paint superstructure.
1952	Paint superstructure.
1955	Replace bituminous pavement.
1959	Replace roller nest at north abutment.
1962	Resurface roadway and sidewalk; new curbing; repair expansion joints; replace 5- foot strips of deck concrete adjacent to curbs; concrete repairs; new scuppers; electrical work.
1963	Paint superstructure. Additional access ladders and platforms, downspouts added to scuppers, repairs to catwalk under deck, replace railing bolts.
1964	10" Welded steel gas main installed beneath deck from abutment to abutment.
1969	Rehabilitate sidewalk and curb; repair substructure cracks.
1970	Door Repair.

Table 2 Cont. - Sagamore Bridge Maintenance and Repair History	
1970	Paint Superstructure.
1974	Repair structural members, concrete, expansion joints, railings; miscellaneous work.
1975	Hanger Cable Replacement.
1976	Joint repair at expansion joint on south abutment.
1981	Remove existing deck and replace with lightweight concrete filled steel grid on galvanized steel stay-in-place forms; add new preformed waterproofing membrane and bituminous concrete wearing surface; new concrete sidewalks and curbs; repair or replace approximately 200 steel gusset/stay plates; replace approximately 1,000 lacing bars; replace approximately 1,000 deteriorated rivets with new high strength bolts; place new deck joints; replace hanger cables; install suicide deterring fence; paint superstructure.
1982	Door Replacement.
1986	Patch spalls and inject cracks on abutments, piers and parapets.
1987	Remove existing bituminous pavements, waterproofing membrane, and upper 1 1/2" of concrete from abutment deck surface; place new 3 1/2" microsilica concrete overlay and wearing surface.
1990	Paint Superstructure.
1996	Replace deck joint between south abutment and Span 3 with modular type expansion joint.
1997	Replace deck joint between north abutment and Span 2 with modular type expansion joint.
1999	Paint superstructure.
2000	Repair concrete abutments and piers; replace deteriorated catwalk grating.
2007	Replaced modular joint between South Abutment and Span 3.
2008	Minor maintenance repairs to catwalk.
2010	Installation of new bearing anchor bolt covers at both abutments.
2010	Repaving of full width roadway (Rosphalt) and resurfacing of sidewalk for Spans 1, 2 and 3 as well as for full length of both abutments. Replaced sidewalks and parapets on both abutments.
2012	Steel repairs throughout the entire length of the bridge including gusset plate patch plates, repairs to lateral bracing and sway bracing and their connections, replacement of missing rivets with bolts at member connections and lacing bar connection, removal of fatigue sensitive weld details on truss members, floorbeams and stringers and replacement of deck drainage support brackets with new drainage downspouts. $2.9 million (combined with Bourne Bridge Steel Repairs – Total $9.7 million)
2014	Painted superstructure. Currently ongoing; work to be completed in 2014. $13.0
2018	Replaced modular joint system & all supporting concrete at south abutment joint; replaced all compression seal joints. $1.7 million

3. FACTORS DEFINING THE NEED FOR REHABILITATION OR REPLACEMENT

The overall condition of both the Bourne and Sagamore bridges is becoming worse as the bridges age and major maintenance projects becomes more frequent. As the condition deteriorates, this leads to the bridges becoming structurally deficient. B oth bridges are functionally obsolete and are routinely unable to provide an efficient flow of traffic in conjunction with the current State and local roadway network leading to the bridge approaches.

a. Structurally Deficient & Functionally Obsolete Criteria

Bridges are considered "structurally deficient" if significant load-carrying elements are found to be in poor or worse condition due to deterioration and/or damage. A "deficient" bridge typically requires maintenance and repair and eventual rehabilitation or replacement to address deficiencies. To remain open to traffic, structurally deficient bridges are often posted with reduced weight limits that restrict the gross weight of vehicles using the bridges. If unsafe conditions are identified during a physical inspection, the structure could be closed.

Bridges are considered functionally obsolete when the geometry of the roadway no longer meets today's minimum design standards for either width or vertical clearance for that roadway classification. A functionally obsolete bridge is one that was built to standards that are not used today. Functionally obsolete bridges are those that do not have adequate lane widths, shoulder widths, or vertical clearances to serve current traffic demand, or those that may be occasionally flooded.

Note, the Federal Highway Administration (FHWA) no longer uses the term "functionally obsolete" to define bridges, however, USACE is using this term for historical context within the framework of the Major Rehab study.

The criteria for defining a "structurally deficient" bridge includes when the condition rating for various bridge elements is considered poor. These bridge elements include the deck, superstructure, or substructure. Any one of these considered to be in poor condition leads to a designation of "structurally deficient".

The previous criteria for defining a "functionally obsolete" bridge includes things such as the deck geometry and approach roadway configurations. Again, while this term is no longer used by FHWA, it is used within this report as a means of identifying obsolete design parameters.

The definitions associated with the terms "structurally deficient" and "functionally obsolete" are based on specific coding of various items in the bridge inventory database for each bridge.

The Bourne Bridge is currently structurally deficient, while the Sagamore Bridge has been historically found to be deficient multiple times in the past. These deficiencies are because certain bridge elements were considered to be in poor condition, based on the condition ratings provided during the inspections of the bridges, as explained in paragraph "4. PRESENT CONDITION OF BRIDGES". Both the Bourne and Sagamore bridges are considered functionally obsolete.

b. Overview Of National Bridge Inspection Program & Condition Ratings

The Bourne and Sagamore Bridges are inspected every 24 months according to the current National Bridge Inspection Standards (NBIS). The NBIS sets the national standards for the proper safety inspection and evaluation of all highway bridges in accordance with 23 U.S.C. 151.

These standards define the organizational responsibilities, qualifications, inspection frequency, procedures, and bridge inventory reporting requirements. The NBIS regulations apply to all publicly owned highway bridges longer than twenty feet located on public roads.

The primary purpose of the NBIS is to locate and evaluate existing bridge deficiencies to ensure the safety of the traveling public. To provide further guidance, the Federal Highway Administration (FHWA) publishes the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*.

The coding guide has been prepared for use in recording and coding the data elements that comprise the National Bridge Inventory (NBI) database. Bridge inspections consist of applying condition ratings to the various bridge components. The coding guide outlines the specific bridge components that are required to be inspected and provides the guidelines on how to apply condition ratings.

Condition ratings are assigned on a scale of 0–9 to the individual components by bridge inspectors using the guidelines established by the FHWA in the coding guide. See Table A-3 - National Bridge Inventory Condition Ratings (FHWA-HIF-11042, Bridge Preservation Guide: Maintaining a State of Good Repair using Cost Effective Investment Strategies, August 2011).

The "Condition Rating" codes for each bridge are obtained from Table A-3 based on the most recent physical inspection of the bridge.

In order to promote uniformity between bridge inspectors, these guidelines are used to rate and code Items 58 (Deck), 59 (Superstructure), and 60 (Substructure). Condition ratings are used to describe the existing, in-place bridge as compared to the as-built condition and to determine structural deficiency ~~and functional obsolescence~~. Evaluation is for the materials and physical condition of the deck, superstructure, and substructure components of a bridge.

It is important to understand that condition codes are properly used when they provide an overall characterization of the general condition of the entire component being rated ("Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges", Report No. FHWA-PD-96-001, December 1995.)

Table A-3 - National Bridge Inventory Condition Ratings

Code	Description	Commonly Employed Feasible Actions
9	EXCELLENT CONDITION	Preventive Maintenance
8	VERY GOOD CONDITION No problems noted.	
7	GOOD CONDITION Some minor problems.	
6	SATISFACTORY CONDITION Structural elements show some minor deterioration.	Preventive Maintenance; and/or Repairs
5	FAIR CONDITION All primary structural elements are sound but may have some minor section loss, cracking, spalling or scour.	
4	POOR CONDITION Advanced section loss, deterioration, spalling or scour.	Rehabilitation or Replacement
3	SERIOUS CONDITION Loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.	
2	CRITICAL CONDITION Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored the bridge may have to be closed until corrective action is taken.	
1	IMMINENT FAILURE CONDITION Major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.	
0	FAILED CONDITION Out of service - beyond corrective action.	

4. PRESENT CONDITION OF BRIDGES

a. Bourne Bridge Condition (2016)

The Bourne Bridge is both structurally deficient and functionally obsolete.

The deck (Item 58) is in fair condition with a condition rating of 5. The superstructure (Item 59) is in poor condition with a condition rating of 4, and the substructure (Item 60) is in good condition with a condition rating of 7. A history of these condition ratings is shown in Figure A-4-1 at the end of this section.

The condition of the deck was downgraded from a previous inspection in 2012 from good to fair due to continuing deterioration of the deck in the abutment spans. Despite recent steel repairs and the removal of fatigue sensitive detail welds, the superstructure remains in poor condition due to continuing deterioration of truss joint gusset plates. The substructure remains in good condition. The most significant inspection findings from the 2016 Routine Inspection that warrant condition codings of fair for the deck, poor for the superstructure and good for the substructure are as follows:

- ***Deteriorated area of deck over the abutments*** - There is a 2'-6" wide by 4'-4" long area of the top of deck in the right southbound lane adjacent to the northern deck joint of the north abutment with full depth spalling of the wearing surface which exposes a similar sized area of sound alligator cracked deck (see Photo 1). The underside of the deck and T-beam below this area exhibit heavy efflorescence and active water leaking during rain. There are four locations in the southbound lane over the south abutment which exhibit the beginning signs of similar conditions.
- ***Deteriorated deck joints*** – The pier 5 deck joint compression seal is dislodged throughout the width of the northbound lanes. The western 6'-0" of the Transflex deck joint at pier 3 (see Photo 2) is loose with up to 1 1/4" gaps at the anchor nuts and up to 1/2" of deflection and bouncing under live load. The modular deck joint at pier 4 exhibits misalignment between the south edge beam and the adjacent center beam in the right southbound lane with the seal between these two beams partially dislodged for a length of 6'-0". Lastly, the pier 6 deck joint compression seal is dislodged and missing across the full width of the roadway (see Photo 3).
- ***Unrepaired gusset plates with significant section loss -*** There are unrepaired gusset plates at eighteen truss joints that continue to exhibit areas of significant section loss and/or deformation due to pack rust in all spans on both trusses. This includes the following locations:

  <u>West Truss</u>

  - Span 2: The east gusset plate at truss joint L5' exhibits up to 1/2" thick pack rust along the south edge of the truss vertical member with slight deforming of the gusset plate.
  - Span 2: The gusset plates at truss joints U0', U2' and U4' exhibit pack rust with section loss to the exterior gusset plate along the edges of the truss vertical member with deformation of the gusset plate. The exterior gusset plate at truss joint U6' exhibits heavy section loss along the interface with the sidewalk channel. The interior gusset plate at

truss joint L7' exhibits heavy section loss along the top of the lower chord member (see Photo 4).

- Span 3: The exterior gusset plates at truss joints U0 and U2 exhibit pack rust with section loss along the edges of the truss vertical member with deformation of the gusset plate (see Photo 5). The exterior gusset plate at truss joint U1 exhibits heavy section loss along the top edge of the vertical member. The exterior gusset plate at truss joint U6 exhibits heavy section loss along the interface with the sidewalk channel.
- Span 5: The gusset plates at truss joint L7 exhibit heavy section loss along the top of the lower chord member and surrounding the vertical member.
- Span 4: The gusset plates at truss joint L5 exhibit heavy section loss along the full height of both gusset plates along both edges of the truss vertical member.
- Span 6: The gusset plates at truss joint L7 exhibit heavy section loss along the top of the lower chord member.

<u>East Truss</u>

- Span 2: The exterior gusset plate at truss joint U0' exhibits pack rust with section loss along the edges of the truss vertical member with deformation of the gusset plate. The exterior gusset plate at truss joint U6' exhibits heavy section loss along the interface with the sidewalk channel.
- Span 3: The exterior gusset plate at truss joint U0 exhibits pack rust with section loss along the edges of the truss vertical member with deformation of the gusset plate. The exterior gusset plate at truss joint U6 exhibits heavy section loss along the interface with the sidewalk channel (see Photo 6).
- Span 6: The gusset plates at truss joint L3 exhibit heavy section loss along the top of the lower chord member and surrounding the vertical member.

♦ ***Fatigue sensitive details (FSD's) on fracture critical members (FCM's)*** – There are numerous FSD's on FCM's in the form of welded attachments (see Photo 7), weld remnants from removed attachments, weld strikes, cuts in the base metal at locations of removed welds and incomplete weld removal with jagged edges or weld undercutting remaining in the base metal.

♦ ***Fatigue sensitive details on stringers*** – There are cuts in the bottom flanges of stringer S4 between floorbeams FB14' and FB15' and stringer S4 between floorbeams FB15' and FB16' in span 1 due to improper weld removal. There are two locations of remnant welds from removed attachments to stringer S4 between floorbeams FB0' and FB1' in span 2.

♦ ***Deteriorated and partially undermined concrete T-beams*** – Beam BM1 is heavily deteriorated in chamber 1 of the south abutment and in chambers 1, 2 and 3 of the north abutment. Previously noted areas of cracks and delaminations to patched areas of beam BM1 in chamber 1 of the south abutment have spalled to the depth of the concrete cover exposing the lower mat of the reinforcing bars which is heavily rusted and exhibits up to 1/8" section loss to the underside of the bars. Beam BM1 in chamber 1 of the north abutment between the north wall and the intermediate strut exhibits longitudinal hairline cracks on the underside with active water infiltration during rain, a minor spall on the underside and heavy efflorescence on the west face. Beam BM1 in chamber 2 of the north abutment exhibits a full length longitudinal crack along the underside of the beam following the construction joint of a repair where the beam had been widened. In addition, the west face of the widened portion of the beam is delaminated. Beam BM1 in chamber 3 of the north abutment exhibits longitudinal hairline cracks with efflorescence and rust staining in an area where the discharge from a drainage scupper splashes directly on the T-beam. The load bearing area of Beams BM1 and BM3 in chamber 2 of the south abutment are partially undermined (see Photo 8).

- ***Spalled and delaminated abutment walls*** – There are two spalls in the north wall of chamber 2 in the south abutment which partially undermine the bearing area for beams BM1 and BM3, and an additional area of delaminated concrete beneath beam BM5. The south wall of chamber 2 in the north abutment exhibits a similar spall and delamination adjacent to and beneath beam BM3.

Some additional general inspection findings are as follows (see Photos 9 through 16):

The main truss members, as well as floorbeams below deck joints, exhibit pack rust between the riveted built-up component elements. There is associated plate warping and localized section loss on individual components such as lacing bars and batten plates for truss members and web stiffeners and flange plates for floorbeams. The fascia stringers also exhibit localized section loss and some pack rust between their connection angles and the stringer webs. The section loss exhibited by the main truss members, floorbeams and stringers is in the form of pitting that is typically 1/8" deep. As a result of the repainting project that began in 2004 and was completed in 2006, much of the corrosion that was previously noted in past inspection cycles has been arrested and is no longer active.

The suspender cables are in fair condition. The suspender cables are assessed using standard criteria presented in section 1.4.2.2. of the Transportation Research Board's National Cooperative Highway Research Program (NCHRP) Report 534. Several suspender cables exhibiting Stage III corrosion and Stage IV corrosion on the outer wires. Stage III is when the zinc coating at the location of ferrous corrosion is typically almost completely consumed. Random wire cracking is possible during this stage. Stage IV is when the wire surface is generally rough and pitted in these areas and wire section loss such as necking as well as wire cracks and breaks are possible at this stage. The suspenders exhibit small areas of corrosion, up to stage III, on thirteen suspenders and stage IV corrosion on five suspenders, an increase of five additional affected suspenders from the 2012 inspection.

Overall the paint system on the Bourne Bridge is in fair condition; however, numerous localized deficiencies were observed during the 2014 inspection including paint adhesion failure, blasting grit debris left on members throughout the bridge, and areas that were not painted. There are also numerous localized areas of the paint system failure such as full width of the faces of the floorbeams directly below roadway joints; the bearings and truss members and their connections below roadway joints; the stringer ends at the deck joints at truss joints 10 and 10'; and the fascia side of the stringers. The suspender cables were noted to be painted, but their paint condition was poor with several suspender cables exhibiting incomplete painting, residual rusted blasting debris within the wires, as well as around the base of the lower socket, and scrapes along the suspender cables that have damaged the galvanizing system and, therefore, made these elements much more susceptible to corrosion.

The bridge traffic safety features, including the bridge railing, transitions, approach guardrails and approach guardrail ends, do not conform to current AASHTO or MassDOT Specifications. In general, these features are composed of nonstandard configurations and do not conform to current MassDOT standards. Additionally, there are areas with no positive connections at the transitions between approach guardrails and the concrete end posts, and some of the W-beam approach guardrails do not conform to current MassDOT standards. These elements are thus rated as not meeting currently accepted standards.There are no scour issues associated with either pier in the water at the edge of the Canal.

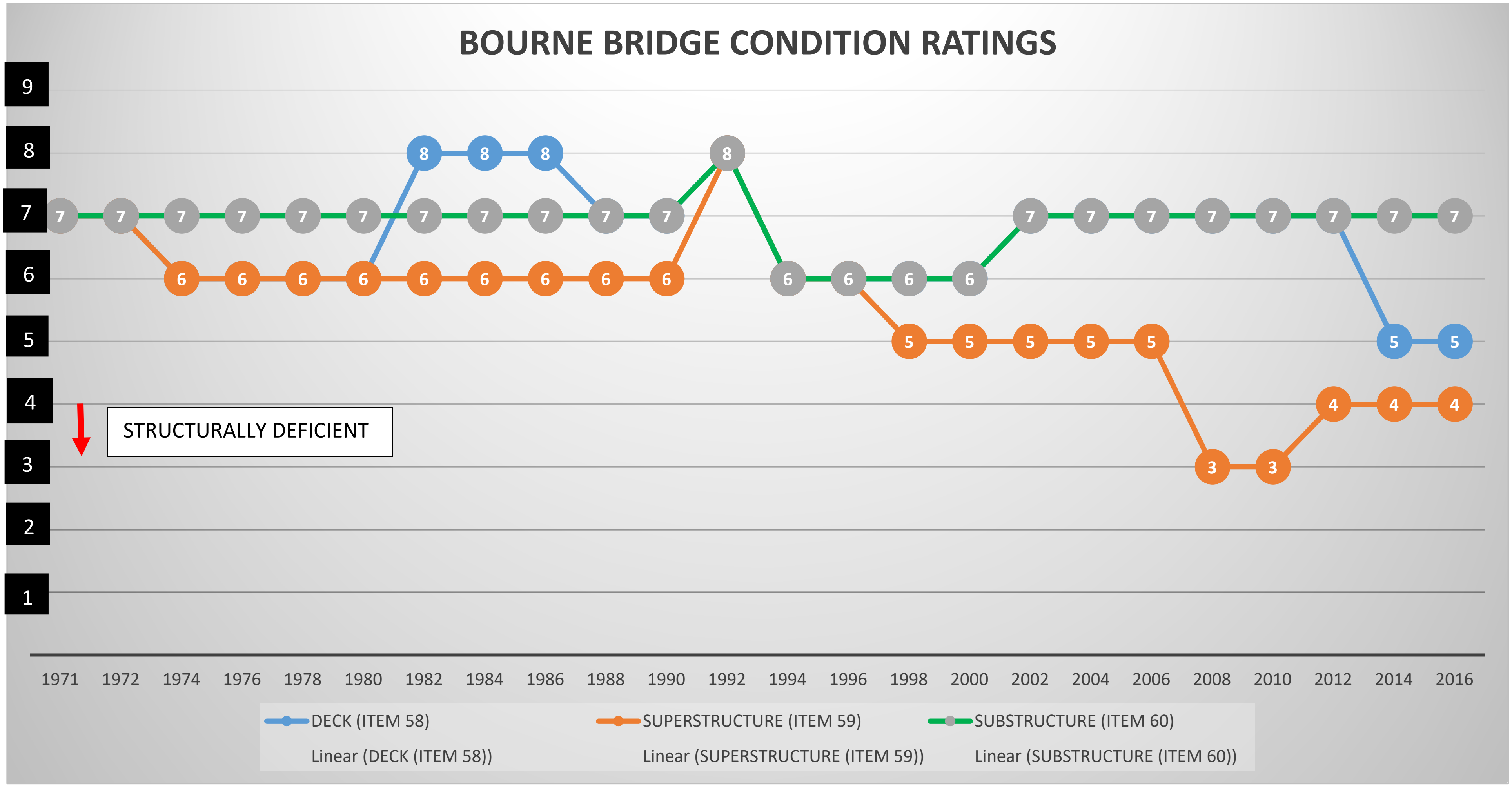


Figure A-4-1: Bourne Bridge History of Condition Ratings

b. Sagamore Bridge Condition (2017)

The Sagamore Bridge is functionally obsolete and has been structurally deficient in the past as recently as 2011. The deck (Item 58), superstructure (Item 59), and substructure (Item 60) are all currently in fair condition with condition ratings of 5. The overall condition of the Sagamore Bridge has not changed since the previous inspection. However, there are individual components that warrant condition ratings of "poor", for example, the gusset plates and other connection plates.

A history of these condition ratings is shown in Figure A-4-2 at the end of this section.

The most significant inspection findings that warrant condition codings of fair for the deck, superstructure and substructure are as follows:

- ♦ ***Deteriorated deck along the reinforced concrete deck joint headers*** **–** Widespread delaminations with localized deep spalls, exposed rebars, and debonded reinforcing in the concrete deck joint headers for the modular deck joints (see Photo 17) located between each abutment and the truss spans. There is vertical misalignment resulting in an uneven riding surface and heavy vehicle impact to span 3 when traveling north. The south modular joint was replaced in 2018.
- ♦ ***Deteriorated truss span deck along exterior stringers*** **–** Shallow spalling of the reinforced concrete in areas where previously deteriorated stay-in-place forms had been removed and the exposed concrete painted.
- ♦ ***Deteriorated abutment span deck*** **–** Widespread hairline map cracking with efflorescence in the underside of the deck throughout all abutment chambers (see Photo 18).
- ♦ ***Gusset plates with significant section loss*** **-** There are gusset plates at twenty-five truss joints that continue to exhibit areas of significant section loss and/or deformation due to pack rust as follows:

  East Truss

  - Span 3: The gusset plates at truss joints U0, U2 and U4 exhibit pack rust with section loss along the edges of the exterior gusset plate with deformation of the gusset plate. The interior gusset plate at L7 exhibits heavy section loss along the top of the lower chord member (see Photo 19).
  - Span 2: The gusset plates at truss joints U0', U2' and U4' exhibit pack rust with section loss along the edges of the exterior gusset plate with deformation of the gusset plate (see Photo 20). The interior gusset plates at U6', L1', L3', and L7' all exhibit heavy section loss and deformation.

  West Truss

  - Span 3: The exterior gusset plates at truss joint U0 and U2 exhibit pack rust with section loss along the edges and deformation of the gusset plate (see Photo 21).
  - Span 1: The south edge of the interior gusset plate at truss joint U11 is bowed 1/4".
  - Span 2: The gusset plates at truss joints U0', L1', U2' and U4' exhibit pack rust with section loss to the exterior gusset plate with deformation of the gusset plate. The interior

gusset plate at U6' exhibits heavy section loss along the interface with the sidewalk channel.

- ***Bearings at the south abutment*** – The south abutment bearings are both near the end of their thermal expansion range. The northwest anchor bolts of both bearings are bent 1/4" out of plumb and exhibit active corrosion (see Photo 22).
- ***Fatigue sensitive details (FSD's) on fracture critical members (FCM's)*** – There are numerous identified FSD's on FCM's in the form of welded attachments, weld remnants from removed attachments, cuts in the base metal at locations of removed welds and incomplete weld removal with jagged edges or weld undercutting remaining in the base metal (see Photo 23).
- ***Fatigue sensitive details on stringers*** –There are four locations of welded repair plates, welded connections and welded attachments to stringer bottom flanges which are considered fatigue sensitive.
- ***Deteriorated and partially undermined concrete T-beams*** – Beam BM4 in chamber 3 of the south abutment exhibits an area of deep scaling and honeycombing. The bearing areas of Beams BM1 and BM5 in chamber 1 of the south abutment and beam BM1 in chamber 1 of the north abutment are partially undermined due to spalling. Beam BM1 in chamber 1 of the north abutment exhibits scattered full width delaminations throughout the underside of the beam.
- ***Spalled and delaminated abutment walls*** – There is a horizontal crack with deep spalls and delaminations scattered along the length of the crack on exterior face of the south abutment just below the parapet and directly over the westbound lane of Sandwich Road, posing a falling debris hazard (see Photo 24). There are two deep spalls in the north wall of chamber 1 in the south abutment which partially undermine the bearing area for beams BM1 and BM5. There are areas of delaminated concrete patches beneath beams BM3 and BM6.

Some additional general inspection findings were as follows (see Photos 25 through 32):

The main truss members exhibit pack rust between the riveted built-up component elements. There is associated plate warping and localized section loss on individual components such as lacing bars and batten plates for truss members. The fascia stringers exhibit localized section loss and some pack rust between the floorbeam connection angles and the stringer webs. The floorbeams exhibit localized section loss to the web and flanges, particularly at the ends. The section loss exhibited by the main truss members, floorbeams and stringers is in the form of pitting that is typically 1/16" to 1/8" deep with localized areas of greater than typical section loss. As a result of the recent repainting project, a majority of the corrosion that was previously noted in past inspection cycles has been arrested and is no longer active.

The suspender cables are in fair condition with several suspender cables exhibiting localized areas of Stage III and Stage IV corrosion on the outer wires; however, these areas are isolated and do not result in any significant section loss.

There is vertical misalignment of the roadway between the south abutment span and span 3 which results in heavy impact, deflection and vibration of span 3 from vehicles travelling north. The drop off from the south abutment span to span 3 is so pronounced that vehicles often bottom out, resulting in scrapes in the wearing surface.

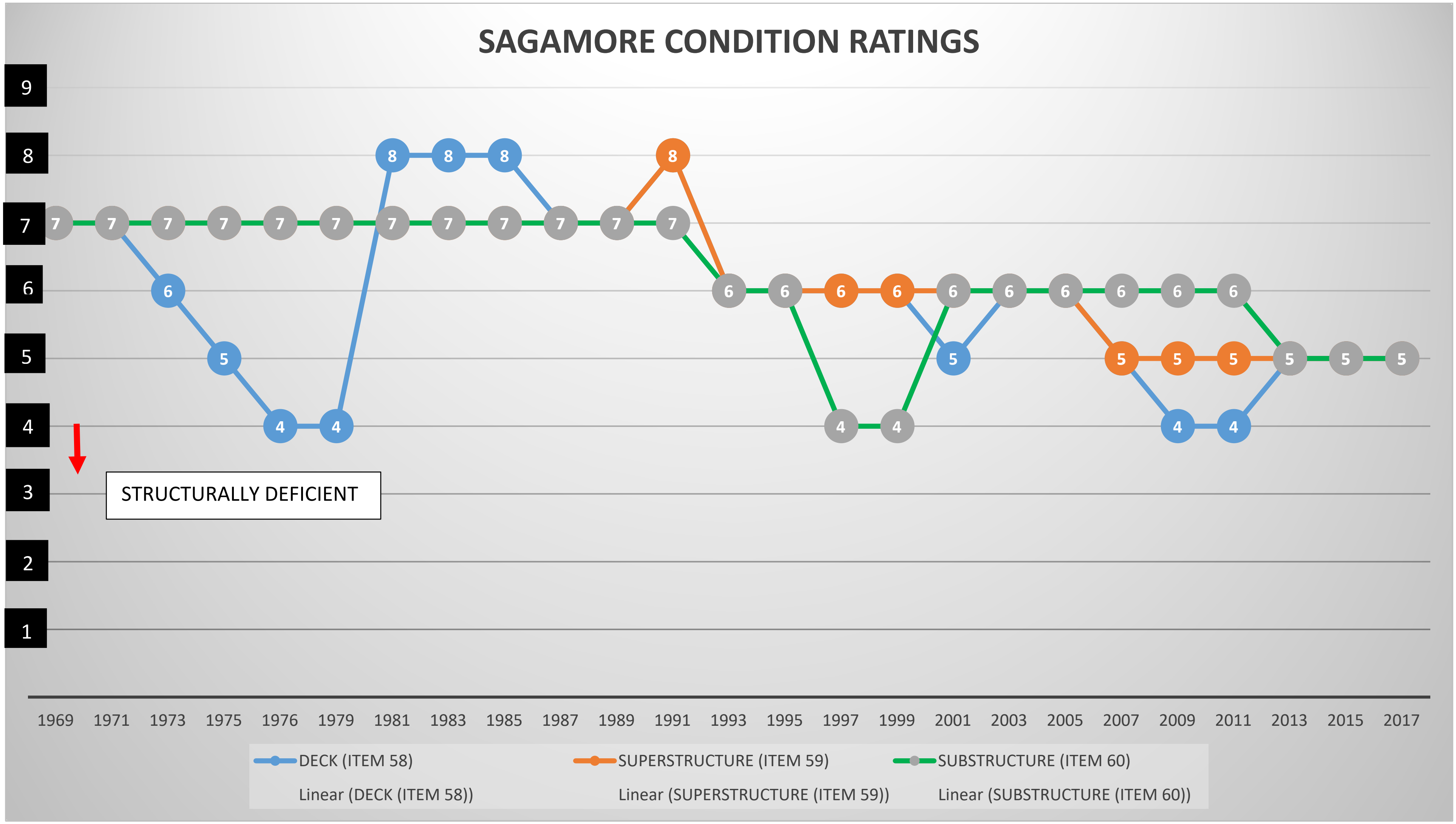


Figure A-4-2: Sagamore Bridge History of Condition Ratings

The bridge traffic safety features, including the bridge railing, transitions, approach guardrails and approach guardrail ends, do not conform to current AASHTO or MassDOT Specifications.

In general, these features are composed of nonstandard configurations and do not conform to current MassDOT standards. Additionally, there are areas with no positive connections at the transitions between approach guardrails and the concrete end posts, and some of the W-beam approach guardrails do not conform to current MassDOT standards. These elements are thus rated as not meeting currently accepted standards.

There are no scour issues associated with either pier in the water at the edge of the Canal.

5. FATIGUE ANALYSIS SUMMARY

As part of this Engineering Reliability Analysis, a load-induced fatigue analysis was conducted in accordance with current AASHTO standards and criteria (*LRFD Bridge Design Specifications* (LRFD) and the *Manual for Bridge Evaluation* (MBE)). The fatigue analysis was conducted for truss members, floorbeams, and stringers. The fatigue analysis results indicated that all primary load carrying members of the truss or flooring system (floorbeams, stringers, etc.) have an infinite fatigue life.

The force effect considered for the analysis consisted of the live load stress range. Only the live load plus dynamic load allowance was considered when computing the stress range cycle; permanent load did not contribute to the stress range.

The *Manual for Bridge Evaluation* states that "Bridges fabricated prior to the adoption of AASHTO's *Guide Specifications for Fracture-Critical Non-redundant Steel Bridge Members* (1978) may have lower fracture toughness levels than are currently deemed acceptable." Destructive material testing to ascertain actual toughness levels of the Bourne & Sagamore bridges has not been conducted. This would likely occur if a Major Rehabilitation of either bridge was found to be warranted.

The fatigue life of a steel bridge detail generally consists of crack initiation and stable crack propagation. The propagation stage continues until the crack reaches a critical length associated with unstable, rapid crack extension, namely fracture.

Fracture toughness reflects the tolerance of the steel for a crack prior to fracture. Fracture of steel bridges is governed by the total stress, including the dead-load stress, and not just the live-load stress range as is the case with fatigue. Older bridges, such as the Bourne and Sagamore bridges which have a satisfactory performance history, likely have adequate fracture toughness for the maximum total stresses that they have experienced.

a. Section Properties

Section moduli were obtained from the Load Rating and Analysis Report, Bourne and Sagamore Highway Bridges, June 2009, conducted by Parsons Brinckerhoff, and updated with the most recent bridge inspection reports. This included the main truss members, floorbeams and stringers. Net section properties of these members were then used to determine the stress ranges for the members.

b. AASHTO Stress Categories

The following categories for load-induced fatigue were investigated. All categories were matched to the most appropriate detail categories in LRFD Table 6.6.1.2.3-1—Detail Categories for Load-Induced Fatigue in order to obtain the threshold $(\Delta F)_{TH}$ parameters.

Table A-4 - Fatigue Details*

MEMBER NAME	AASHTO STRESS CATEGORY	DESCRIPTION (INSTALLATION DATE)
STRINGER	A	Rolled member, typical stringer at midspan (1935).
FLOORBEAM	D	Bottom flange at floorbeam at net section of riveted connections (1935).
FLBM – Welded Stiffener Repairs	E’	Plate welded to vertical leg of floorbeam bottom flange angle (1964). Welded floorbeam web stiffener repair welded to floorbeam bottom flange.
FLBM - End Flbms. - remnant drain trough welds; Welded gas main bracket	E	Utility pipe support angle welded to floorbeam web (1964). Stub plate continuously welded to floorbeam web (1962). CRACKED ONES REPAIRED 2013; OTHERS REMAIN.
TRUSS	D	Truss member at net section of riveted connection (1935).

*Note, a complete list of Fatigue Details is contained in the latest inspection reports for each bridge.

c. Truss Members

The truss chords and diagonals are built-up riveted members comprised of angles, channels, and plates fabricated into box members using lacing bars. The basic riveted members are assigned a fatigue resistance of category D, which is also more conservative. No cracking exists in any of the truss members.

The Constant Amplitude Fatigue Limit (CAFL) for category D is 7.0 ksi. The CAFL is the stress range that below which no fatigue crack growth would be expected. In other words, if all live load stress-range cycles were kept below the CAFL, no fatigue cracking should occur and the member or detail would be expected to have an infinite life. The riveted truss members all have a stress range below the CAFL and, therefore, have an infinite calculated fatigue life.

d. Stringers

The stringers are all rolled members. According to LRFD Table 6.6.1.2.3-1, the stringers have a fatigue category of A and a CAFL of 24 ksi. No cracking exists in any of the stringers. The stringers all have a stress range below the CAFL and, therefore, have an infinite calculated fatigue life. However, there are five fatigue sensitive details (welded attachments) located on some stringers that have a calculated finite fatigue life ranging from 3 years to 28 years. These details show no signs of cracks and are monitored every 24 months. In addition, these details are scheduled to be removed in the next steel repair contract or if a major rehab is undertaken.

e. Floorbeams

The floorbeams are built-up riveted members made up of a web plate and flange angles. A cover plate is located at mid-span of the floorbeam. The floorbeams are category D details (rivets) and have category E details (welded attachments).

The welds on these members were added as part of the previous installation of drainage components and as well as other miscellaneous attachments. It is noted that some of the welds placed on the web plate directly connect the angles to the web plate. These welds only appear to be located at end floorbeams (PP 0, 0', 10 and 10') where the original drainage system was installed. These welds provide a direct path for cracks to travel from one component to another, should they occur.

There are several details on the floorbeams that are of concern in terms of the fatigue limit state. The riveted members, in and of themselves, are category D details. However, due to the addition of various welded attachments, details with lower fatigue resistance have been placed on the floorbeams. Although these welded details have lower fatigue resistance, they are not all located in regions of high stress range (*e.g., the angle welded to the floorbeam web supporting the gas line is nearly at the neutral axis*).

The welds used to attach the gas-line support bracket to the floorbeam web are somewhat more straightforward in terms of their assessment. The longitudinal length of the weld used to attach the bracket (fabricated from a rolled angle) to the web determines if the joint is classified as a category C, D, or E detail. If there is a weld placed on the horizontal leg of the support angle, the detail will be considered category E, since the length of the weld will be greater than four inches.

With the exception of the end floorbeams, all floorbeams are internally redundant since they are built-up riveted members. Hence, a crack in one flange component does not have a direct path into the others.

According to LRFD Table 6.6.1.2.3-1, the floorbeams have a fatigue category of D and a CAFL of 7 ksi. No cracking exists in any of the floorbeams. The floorbeams all have a stress range below the CAFL and, therefore, have an infinite calculated fatigue life.

The floorbeams with various fatigue sensitive details that are categorized as E (CAFL of 4.5 ksi) and E' (CAFL of 2.6 ksi) have a finite fatigue life. Current estimates of remaining fatigue life (based on previous fatigue life calculations) range from about 140 years to over 500 years for these details. However, some of these fatigue sensitive details are not located in areas of high stress (at or near the neutral axis of the member) and these details have shown no signs of cracks due to fatigue.

All of the fatigue sensitive details are routinely monitored for cracks. In addition, these details are scheduled to be removed in the next steel repair contract or if a major rehab is undertaken.

f. Traffic

The latest Annual Average Daily Traffic (AADT) was obtained from MassDOT's permanent traffic counting stations nearest the bridges. The most recent 10-year average was chosen for the analysis at both bridges. The AADT for the Bourne Bridge was 44,447 and the Sagamore Bridge was 51,756.

Based on LRFD Table C3.6.1.4.2-1, the fraction of trucks in traffic for a highway classified as 'Urban Interstate' is 0.15. Using these data, the present average number of trucks per day for all directions of truck traffic $[ADTT]_{PRESENT}$ was computed. The $[ADTT]_{PRESENT}$ was used to estimate the total fatigue life of the previously described truss and floor system members and details.

g. Live Load

The load applied for fatigue analysis comprises the HL-93 design truck with a fixed rear axle spacing of 30 feet between the 32-kip axles. The 30-foot rear axle spacing represents an average axle spacing as opposed to the variable spacing used for design purposes. Fatigue is not based upon a single one-off load, but on the vast majority of average trucks crossing the bridge.

i) Distribution factor

LRFD 3.6.1.4.3 states that the distribution factor (DF) to be used to approximate the load distribution shall be the DF for one-traffic lane. Distribution factors were obtained from the load rating conducted by Parsons Brinckerhoff. The lever rule was used to obtain the controlling DF of 0.84.

ii) Dynamic load allowance

A dynamic load allowance of 15% is applied to the truck, representing average conditions. LRFD 3.6.2 specifies a dynamic load allowance for the Fatigue Limit State of 15%.

iii) Live Load Moment

In order to determine the maximum live load moment at the member of interest, the fatigue truck was applied to the truss models previously developed by Parsons Brinckerhoff for the load rating.

h. Infinite Life Check

The infinite-life check of all fatigue prone details was performed in accordance with MBE 7.2.4. In theory, a fatigue-prone detail will experience infinite life if the stress range at that particular detail is below a constant amplitude fatigue threshold, $(\Delta F)_{TH}$. If the stress range of the member or detail exceeds the threshold, the total fatigue life should be estimated.

i. Estimating Finite Fatigue Life

Certain fatigue sensitive details on both the stringers and the floorbeams indicated a finite fatigue life, but these specific fatigue sensitive details are monitored every 24 months during each routine inspection. In addition, these specific FSD's would be remediated either during a major rehabilitation project or the next scheduled steel repair contract prior to any specific rehabilitation project.

Also, although no site-specific stress measurements have been obtained for either the Bourne or Sagamore bridges, calculated fatigue stress ranges can overestimate the actual in-service stress ranges. Often, this is due to unaccounted stress redistribution among structural components, simplifications in structural analysis models, and inaccurate load models.

j. Fatigue Summary

The primary load carrying members comprised of the riveted truss members, rolled stringers, and built-up floorbeams all have an infinite calculated fatigue life. FSD's that are categorized as E and E' on the floorbeams having a finite fatigue life and FSD's located on certain stringers with category D or E welds. Some of these fatigue sensitive details are not located in areas of high stress and these details have shown no signs of cracks due to fatigue. Regardless, all of the fatigue sensitive details on the trusses, floorbeams or stringers are routinely monitored for cracks.

6. CORROSION ANALYSIS SUMMARY

A corrosion analysis was conducted to aid in determining the overall long-term impact of corrosion on various bridge members, including the trusses, floorbeams, stringers and gusset plates, in relation to load rating factors over the 50-year study period.

For example, the rating factor of a truss member which is currently above 1.0, could possibly become less than 1.0 after accounting for corrosion over a 50-year study period. This is a factor which could lead to the possibility of needing to post the bridge at some point within that 50-year time period. Rating factors below 1.0 are only an indicator of posting (i.e. legally reducing the weight of vehicles permitted to cross the bridge), since a detailed load rating analysis for all members of these bridges was not conducted for this study.

a. Rate of Corrosion

An analysis of the rate of corrosion was accomplished for this study. Corrosion rates for this study were determined from measurements taken on fascia stringers of the Sagamore Bridge and on truss members and gusset plates of both the Sagamore and Bourne Bridges. All of these members are original steel members comprised of silicon steel. Silicon steel was used in various members of the trusses of these bridges due to its relatively high strength. The addition of silicon to steel contributes to the strength and hardness of the material.

The ISO Standard 9223 (Reference p) is widely used outside the U.S. for classification of environmental corrosivity. This reference defines various service environment categories. This standard breaks down into corrosivity categories from C1 (mild) to C5 (severe) with an additional category, C5M (severe marine) for marine exposures. The expected range of corrosion rate for each classification is shown in Table A-5. While this standard is not widely used in the highway bridge industry in the U.S., it has gained popularity for offshore and utility structures and an increasing number of coatings suppliers and researchers are referring to this classification system for generating performance data and recommending materials.

From Figure A-6-1, the average rate of corrosion for the Sagamore and Bourne Bridges is 0.0027 inches/yr. This rate of corrosion is consistent with Category C4, shown in Table A-6-1 below. The C4 category represents coastal areas with moderate salinity. This rate of corrosion is based on actual measurements of original steel components. These components have received regular maintenance of their coatings during their service life, therefore, it is assumed that this rate of corrosion includes continued maintenance painting of the bridges.

Table A-5 - Carbon Steel Corrosion Rates for Various Environments According to ISO 9223	
Service Environment	Carbon Steel Corrosion Rate (in. per year)
C1 - Very Low	0.00005
C2 - Low	<0.001
C3 - Medium	0.001 to 0.002
C4 - High	0.002 to 0.003
C5I - Very High (Industrial)	0.003 to 0.008
C5M - Very High (Marine)	0.008 to 0.028

Table A-6-1 – Corrosion Rates		
BRIDGE OR LOCATION	**AVG. CORROSION RATE (IN./YR.)**	**SOURCE**
SAGAMORE - STRINGERS	0.0017 - 0.0018	On site - 2013
SAGAMORE – TRUSS MEMBERS	0.0039	REFERENCES g & h
BOURNE – TRUSS MEMBERS	0.0023	REFERENCES i & j
SAGAMORE – GUSSET PLATES	.0027	REFERENCES g & h
BOURNE – GUSSET PLATES	.0027	REFERENCES i & j
AVERAGE	**0.0027**	
MARINE ENVIR., CAPE KENNEDY, FL (0.5 MI, FROM COAST)	0.00162	REFERENCE n, TABLE 1
MARINE ENVIR., CAPE KENNEDY, FL (60 YD. FROM COAST, 60 FT. ELEV.)	0.00241	REFERENCE n, TABLE 1
MARINE ENVIR., CAPE KENNEDY, FL (60 YD. FROM COAST, 30 FT. ELEV.)	0.00279	REFERENCE n, TABLE 1
MARINE ENVIR., KURE BEACH, NC (800 FT. FROM COAST)	0.00335	REFERENCE n, TABLE 1
C3 – COASTAL AREAS WITH LOW SALINITY	0.001 – 0.002	REFERENCES n & p
C4 – COASTAL AREAS WITH MODERATE SALINITY	0.002 – 0.003	REFERENCES n & p

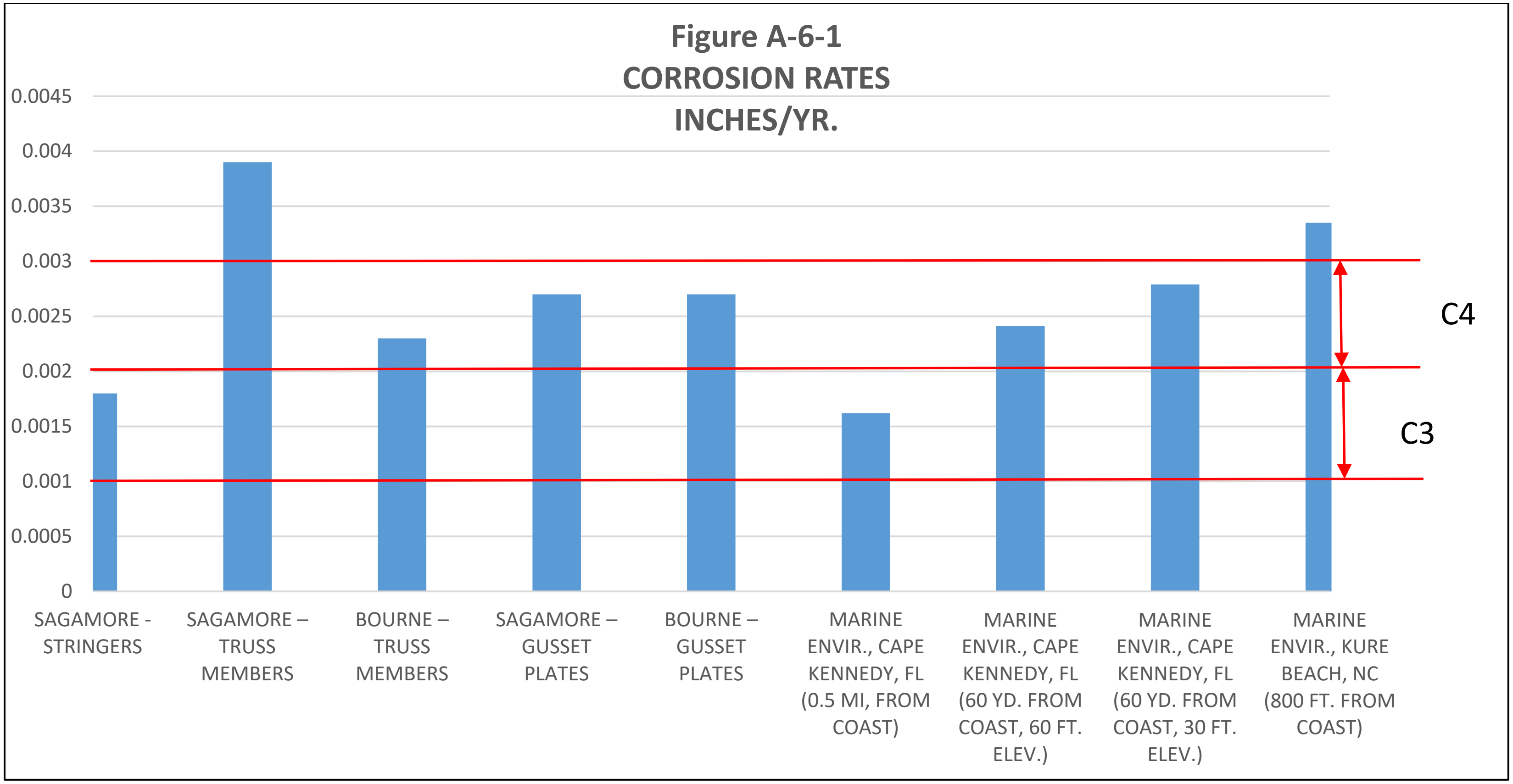


Figure A-6-1
CORROSION RATES
INCHES/YR.
0.0045
0.004
0.0035
0.003
0.0025
0.002
0.0015
0.001
0.0005
0
SAGAMORE - STRINGERS
SAGAMORE – TRUSS MEMBERS
BOURNE – TRUSS MEMBERS
SAGAMORE – GUSSET PLATES
BOURNE – GUSSET PLATES
MARINE ENVIR., CAPE KENNEDY, FL (0.5 MI, FROM COAST)
MARINE ENVIR., CAPE KENNEDY, FL (60 YD. FROM COAST, 60 FT. ELEV.)
MARINE ENVIR., CAPE KENNEDY, FL (60 YD. FROM COAST, 30 FT. ELEV.)
MARINE ENVIR., KURE BEACH, NC (800 FT. FROM COAST)
C4
C3

b. Corrosion Results

The rate of corrosion was applied to critical members such as floorbeams, stringers and gusset plates. The results are summarized below.

i) Floorbeams

The controlling floorbeam rating (HS-20) for each bridge is within Spans 1, 2 and 3. This was revised for each 10-year period of the study (see Table 6-2 below). The current HS-20 controlling floorbeam rating is 0.87 and occurs on both bridges for Spans 1, 2 and 3. By the end of the 50-year study period, assuming a linear rate of corrosion of 0.0027 inches per year applied to the bottom flange angles and plates, this rating is reduced to about 0.74. While a bridge is not posted for HS-20 trucks, it is an indication that as the HS-20 rating lowers due to corrosion, it is also likely that the Massachusetts State legal (posting) loads will also be lowered, leading to eventual posting of the bridge. This is an indication that without either a major rehab or repair contract, the floorbeams will result in the need to place weight restrictions on each bridge for the Massachusetts legal loads in approximately 2036.

Since the floorbeams are currently rated at 0.87 (and no posting is required), it's strictly engineering judgement that by the time the rating factor is around 0.81 or 0.82 in 20 years, a weight restriction (load posting) may be necessary. This would mean potential load limits would be placed on either or both bridges in about 20 years from the date of analysis, or around 2036.

Certainly by the end of the 50-year period, overweight permit loads will routinely be denied from crossing either bridge due to the condition of the floorbeams. Table A-6-3 summarizes the linear assessment of corrosion on the floorbeams over the next 50 years. An elevation of a typical floorbeam with various cross-sections indicated is shown below.

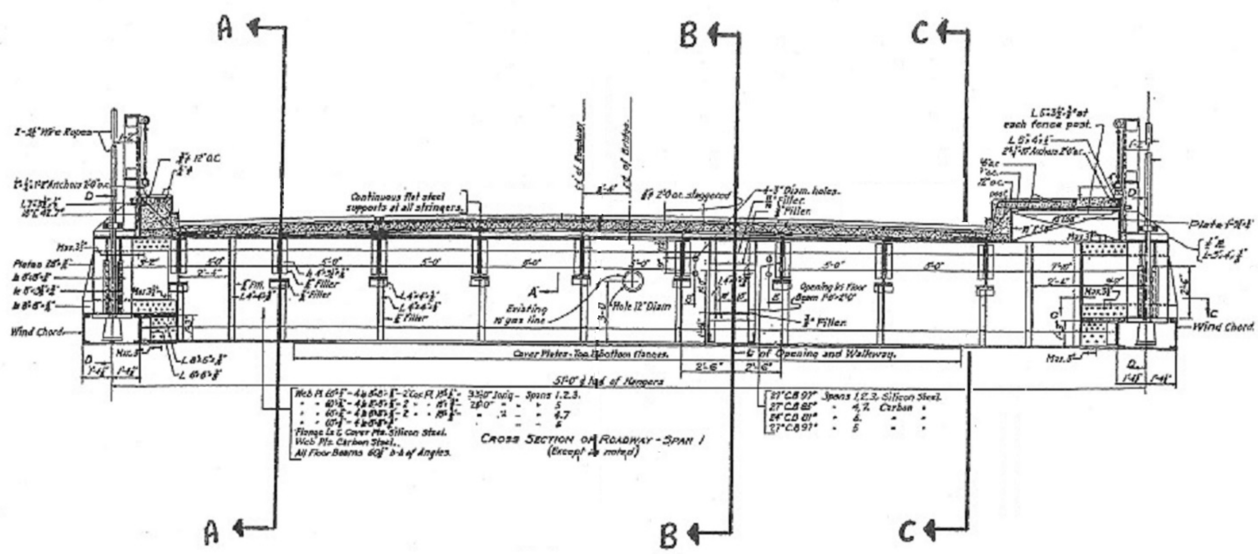


Table A-6-2 – Floorbeam HS-20 Inventory Rating Factors

BRIDGE/SPAN		@ PRESENT	@ T=10 YRS.	@ T=20 YRS.	@ T=30 YRS.	@ T=40 YRS.	@ T=50 YRS.
		0	**10**	**20**	**30**	**40**	**50**
SAGAMORE BRIDGE SPANS 1, 2 & 3							
	SECT. MOD. (IN^3) @ A-A	1202.51	1183.54	1164.51	1145.50	1126.42	1107.37
	SECT. MOD. (IN^3) @ B-B	1746.74	1722.46	1698.14	1673.80	1649.43	1625.04
	SECT. MOD. (IN^3) @ C-C	1202.51	1183.54	1164.51	1145.50	1126.42	1107.37
	HS-20 INV RATING FACTOR	0.87	0.84	0.81	0.78	0.76	0.73
BOURNE BRIDGE SPANS 1, 2 & 3							
	SECT. MOD. (IN^3) @ A-A	1235.44	1216.13	1196.77	1177.42	1158.02	1138.64
	SECT. MOD. (IN^3) @ B-B	1752.65	1728.48	1704.28	1680.05	1655.79	1631.51
	SECT. MOD. (IN^3) @ C-C	1235.44	1216.13	1196.77	1177.42	1158.02	1138.64
	HS-20 INV RATING FACTOR	0.87	0.85	0.82	0.79	0.76	0.74

ii) Stringers

The controlling stringer rating (HS-20) for each bridge was revised for each 10-year period of the study. The current HS-20 controlling stringer rating is 0.96 and occurs at interior stringers on the Bourne Bridge for Spans 1, 2 and 3. Using the previously calculated rate of corrosion of 0.0027" per year and applying this to the stringer rating factors computed by Parsons Brinckerhoff, the stringers in Spans 1, 2 and 3 control the rating factor for stringers.

The interior stringers on both bridges in Spans 1, 2 and 3 will require rehabilitation or replacement in Year 10 for the Bourne Bridge and Year 20 for the Sagamore Bridge. Exterior stringers will require rehabilitation or replacement in Year 30 for both bridges.

This is an indication that without either a major rehabilitation or repair contract, the stringers will result in the need to place weight restrictions on each bridge for the Massachusetts legal loads in approximately 2026 for the Bourne Bridge and 2036 for the Sagamore Bridge. Certainly by the end of the 50-year period, overweight permit loads will routinely be denied from crossing either bridge due to the condition of the stringers.

Table 6-3 summarizes the linear assessment of corrosion on the stringers over the next 50 years.

Table A-6-3 – Stringer HS-20 Inventory Rating Factors

BRIDGE/SPAN		@ PRESENT	@ T=10 YRS.	@ T=20 YRS.	@ T=30 YRS.	@ T=40 YRS.	@ T=50 YRS.
		0	**10**	**20**	**30**	**40**	**50**
SAGAMORE BRIDGE - SPANS 1, 2 & 3							
INTERIOR STRINGER	**SECTION MODULUS (IN^3)**	219.97	215.57	211.26	207.03	202.89	198.84
	HS-20 INV RATING FACTOR	0.99	0.96	0.93	0.90	0.87	0.84
EXTERIOR STRINGER	**SECTION MODULUS (IN^3)**	219.97	215.57	211.26	207.03	202.89	198.84
	HS-20 INV RATING FACTOR	1.06	1.02	0.98	0.95	0.91	0.88
BOURNE BRIDGE – SPANS 1,2 & 3							
INTERIOR STRINGER	**SECTION MODULUS (IN^3)**	215.39	211.08	206.86	202.72	198.67	194.70
	HS-20 INV RATING FACTOR	0.96	0.93	0.90	0.87	0.84	0.82
EXTERIOR STRINGER	**SECTION MODULUS (IN^3)**	216.60	212.27	208.02	203.86	199.78	195.79
	HS-20 INV RATING FACTOR	1.03	0.99	0.96	0.92	0.89	0.85

iii) Gusset Plates

Gusset plates are all non-redundant and are considered fracture critical members (FCM), meaning the failure of one of these elements will likely lead to catastrophic failure of an entire span. Therefore, the importance of gusset plates to the overall structural integrity of the bridges cannot be overstated.

A comprehensive load rating of all the gusset plates was beyond the scope of this investigation. However, using the previous gusset plate load rating performed by Parsons Brinckerhoff in 2011, recent bridge inspection reports, and the linear rate of corrosion of 0.0027 inches per year, a list of priority gusset plates was developed, as shown below in Tables 6-4 and 6-5. These are HS-20 load rating factors, which is a reflection of a reduction in overall load capacity associated with deterioration of these members. It is also an indication of the possible need for future weight restrictions on the bridges.

It is important to note that although one specific location is listed, the assumption is that all similar locations for both the Bourne and the Sagamore bridges will require rehabilitation. This is due to truss symmetry in configuration, similar materials and age, and identical environments causing deterioration and corrosion within both bridges.

Therefore, if a particular location has a low rating factor, it would apply to similar locations on both trusses at both bridges requiring rehabilitation or repair.

At the current rate of corrosion, various main truss gusset plates will likely have rating factors less than 1.0 in ten to twenty years.

Those gusset plates where the fastener shear controlled the rating are not included in this table because fastener shear is more easily rectified by simply replacing the existing rivets with high-strength bolts and is not influenced by the current rate of corrosion. These locations would be repaired during any major rehabilitation or steel repair project.

Tables 6-4 and 6-5 summarize the linear assessment of corrosion on the gusset plates over the next 50 years.

Table A-6-4 – Bourne Bridge Gusset Plate HS-20 Inventory Rating Factors

BRIDGE/SPAN/PANEL POINT	RESISTANCE TYPE	@ PRESENT	@ T=10 YRS.	@ T=20 YRS.	@ T=30 YRS.	@ T=40 YRS.	@ T=50 YRS.
INTERIOR GUSSET PLATES							
BOURNE BRIDGE - SPAN 4							
L2	PLATE THICKNESS (INCHES)	0.5000	0.4730	0.4460	0.4190	0.3920	0.3650
	GROSS SECTION YIELDING (TENSION)	1.26	1.11	0.95	0.80	0.65	0.50
L3	PLATE THICKNESS (INCHES)	0.3750	0.3480	0.3210	0.2940	0.2670	0.2400
	COMPRESSION BUCKLING	1.71	1.41	1.37	1.20	1.03	0.86
BOURNE BRIDGE - SPAN 3							
U2	PLATE THICKNESS (INCHES)	0.7500	0.7230	0.6960	0.6690	0.6420	0.6150
	COMPRESSION BUCKLING	1.08	0.97	0.86	0.76	0.65	0.54
L1	PLATE THICKNESS (INCHES)	0.3750	0.3480	0.3210	0.2940	0.2670	0.2400
	GROSS SECTION YIELDING (TENSION)	1.44	1.28		0.96	0.80	
L3	PLATE THICKNESS (INCHES)	0.3750	0.3480	0.3210	0.2940	0.2670	0.2400
	GROSS SECTION YIELDING (TENSION)	1.41	1.25	1.09			0.61
BOURNE BRIDGE - SPAN 5							
U7	PLATE THICKNESS (INCHES)	0.6250	0.5980	0.5710	0.5440	0.5170	0.4900
	GROSS SECTION YIELDING (SHEAR)	1.00	0.89				0.45
EXTERIOR GUSSET PLATES							
BOURNE BRIDGE - SPAN 4							
L2	PLATE THICKNESS (INCHES)	0.5000	0.4730	0.4460	0.4190	0.3920	0.3650
	GROSS SECTION YIELDING (TENSION)	1.26	1.11	0.95	0.80	0.65	0.50
L3	PLATE THICKNESS (INCHES)	0.3750	0.3480	0.3210	0.2940	0.2670	0.2400
	COMPRESSION BUCKLING	1.71	1.41	1.37	1.20	1.03	0.86
BOURNE BRIDGE - SPAN 3							
U2	PLATE THICKNESS (INCHES)	0.3750	0.3480	0.3210	0.2940	0.2670	0.2400
	COMPRESSION BUCKLING	2.2900	1.400	0.910	0.510	0.200	0.000

Table A-6-5 – Sagamore Bridge Interior Gusset Plate HS-20 Inventory Rating Factors

BRIDGE/PANEL POINT	RESISTANCE TYPE	@ PRESENT	@ T=10 YRS.	@ T=20 YRS.	@ T=30 YRS.	@ T=40 YRS.	@ T=50 YRS.
SAGAMORE BRIDGE							
U0	PLATE THICKNESS (INCHES)	0.4688	0.4418	0.4148	0.3878	0.3608	0.3338
	COMPRESSION BUCKLING	1.50	1.38	1.26	1.14	1.02	0.89
U4	PLATE THICKNESS (INCHES)	0.5625	0.5355	0.5085	0.4815	0.4545	0.4275
	COMPRESSION BUCKLING	1.11	1.00	0.89	0.78	0.67	0.56
U6'	PLATE THICKNESS (INCHES)	0.5000	0.4730	0.4460	0.4190	0.3920	0.3650
	GROSS SECTION YIELDING (TENSION)	2.0300	1.870	1.710	1.550	1.390	1.230
L1	PLATE THICKNESS (INCHES)	0.2500	0.2230	0.1960	0.1690	0.1420	0.1150
	GROSS SECTION YIELDING (TENSION)	1.44	1.28	1.11	0.96	0.80	0.63
L3	PLATE THICKNESS (INCHES)	0.2500	0.2230	0.1960	0.1690	0.1420	0.1150
	GROSS SECTION YIELDING (TENSION)	1.41	1.25	1.08	0.93	0.77	0.61
L7	PLATE THICKNESS (INCHES)	0.5000	0.4730	0.4460	0.4190	0.3920	0.3650
	GROSS SECTION YIELDING (TENSION)	2.05	1.81	1.57	1.33	1.09	0.85
U11	PLATE THICKNESS (INCHES)	0.7080	0.6810	0.6540	0.6270	0.6000	0.5730
	GROSS SECTION YIELDING (SHEAR)	1.13	1.05	0.98	0.90	0.82	0.74
U13	PLATE THICKNESS (INCHES)	0.7500	0.7230	0.6960	0.6690	0.6420	0.6150
	COMPRESSION BUCKLING	1.1600	1.060	0.960	0.860	0.750	0.640
U8	PLATE THICKNESS (INCHES)	0.8750	0.8480	0.8210	0.7940	0.7670	0.7400
	GROSS SECTION YIELDING (SHEAR)	1.40	1.30	1.21	1.11	1.01	0.91
U2	PLATE THICKNESS (INCHES)	0.7500	0.7230	0.6960	0.6690	0.6420	0.6150
	COMPRESSION BUCKLING	1.93	1.82	1.72	1.61	1.51	1.40

Table A-6-5 (CONT.) – Sagamore Bridge Exterior Gusset Plate HS-20 Inventory Rating Factors

BRIDGE/PANEL POINT	RESISTANCE TYPE	@ PRESENT	@ T=10 YRS.	@ T=20 YRS.	@ T=30 YRS.	@ T=40 YRS.	@ T=50 YRS.
SAGAMORE BRIDGE							
U0	PLATE THICKNESS (INCHES)	0.3750	0.3480	0.3210	0.2940	0.2670	0.2400
	COMPRESSION BUCKLING	2.17	1.94	1.70	1.45	1.19	0.91
U4	PLATE THICKNESS (INCHES)	0.3750	0.3480	0.3210	0.2940	0.2670	0.2400
	COMPRESSION BUCKLING	3.00	2.71	2.41	2.10	1.76	1.39
U6'	PLATE THICKNESS (INCHES)	0.7500	0.7230	0.6960	0.6690	0.6420	0.6150
	GROSS SECTION YIELDING (TENSION)	3.49	3.33	3.17	3.02	2.86	2.70
L1	PLATE THICKNESS (INCHES)	0.3130	0.2860	0.2590	0.2320	0.2050	0.1780
	GROSS SECTION YIELDING (TENSION)	1.81	1.65	1.49	1.33	1.17	1.01
L3	PLATE THICKNESS (INCHES)	0.2500	0.2230	0.1960	0.1690	0.1420	0.1150
	GROSS SECTION YIELDING (TENSION)	1.41	1.25	1.08	0.93	0.77	0.61
L7	PLATE THICKNESS (INCHES)	0.3125	0.2855	0.2585	0.2315	0.2045	0.1775
	GROSS SECTION YIELDING (TENSION)	2.05	1.81	1.57	1.33	1.09	0.85
U11	PLATE THICKNESS (INCHES)	0.7080	0.6810	0.6540	0.6270	0.6000	0.5730
	GROSS SECTION YIELDING (SHEAR)	1.13	1.05	0.98	0.90	0.82	0.74
U13	PLATE THICKNESS (INCHES)	0.7500	0.7230	0.6960	0.6690	0.6420	0.6150
	COMPRESSION BUCKLING	1.16	1.06	0.96	0.86	0.75	0.64
U8	PLATE THICKNESS (INCHES)	0.8750	0.8480	0.8210	0.7940	0.7670	0.7400
	GROSS SECTION YIELDING (SHEAR)	1.40	1.30	1.21	1.11	1.01	0.91
U2	PLATE THICKNESS (INCHES)	0.3750	0.3480	0.3210	0.2940	0.2670	0.2400
	COMPRESSION BUCKLING	3.9600	3.620	3.250	2.850	2.410	1.890

c. Corrosion Summary

This simplified linear corrosion analysis found that various main truss gusset plates will likely have rating factors less than 1.0 in ten to twenty years. This means that significant costs will need to be incurred to replace or rehabilitate these gusset plates within the 50-year study period in order to prevent the bridges from being posted for weight restrictions.

Both the stringers and the floorbeams will have reduced capacity and may result in the need to initiate weight restrictions on both bridges in 20 to 30 years, or less. It is also likely that overweight permits will also need to be restricted when the bridge is load posted.

7. ALTERNATIVES

There are three alternatives which have been investigated.

1) Base Condition
2) Major Rehabilitation
3) Bridge Replacement

a. Base Condition

This alternative is synonymous with a "without project" condition, and assumes that the bridges will continue to be operated efficiently and with due diligence for vehicular and marine safety. In the event of unsatisfactory performance of a bridge component, it is assumed that emergency funding will be made available to address the deficiency. This scenario portrays a condition where the reliability of the bridges is allowed to fall below the current condition, but that the bridge remains functional. For the Base Condition alternative, the following items would continue to be maintained or repaired on the three main areas of the bridges:

<u>Superstructure</u>
1. Advanced deterioration of secondary member, non-critical gusset plate, stringer, floorbeam, or hanger cable.
2. Advanced deterioration of main truss member or critical gusset plate.
3. Catastrophic damage to main truss member or critical gusset plate.

<u>Deck</u>
1. Localized deterioration of roadway joint(s), granite curbs, concrete-filled steel grid over bridge spans, or reinforced concrete deck at the abutments.
2. Widespread deterioration of concrete-filled steel grid deck over bridge spans and reinforced concrete deck at abutments.

<u>Substructure</u>
1. Localized concrete defects such as cracks or spalls on vertical surfaces of piers or degradation of concrete under bearings on piers.
2. Widespread concrete defects such as cracks or spalls on vertical surfaces of piers or degradation of concrete under bearings on piers.

b. Major Rehabilitation

Major rehabilitation items are large scale projects including structural improvements that are performed less frequently, outside of the purview of normal maintenance, and are aimed at prolonging the service life of the bridge, maintaining an acceptable load carrying capacity, and preserving overall public safety on the structure. Contractors are generally hired for these major work items since the work required is somewhat specialized in most cases. It is estimated that a full rehabilitation project would take 4 years for each bridge. Assuming a project start in 2021, the rehabilitation of both bridges would extend to 2029, as working on both bridges concurrently would present an unacceptable impact to the region.

This scenario assumes that all known structural deficiencies on both bridges will be addressed under a major rehabilitation contract. This would include multiple large projects undertaken in successive construction seasons in order to provide a comprehensive rehabilitation of both the Bourne and Sagamore Bridges.

Minimizing traffic congestion and impacts during a Major Rehabilitation requires keeping at least one of the bridges open with no traffic control at any given time. Therefore, in order to provide sufficient traffic capacity and lessen adverse impacts to traffic throughout the duration of a major rehabilitation project, only one bridge at a time can be worked on, thus extending the overall timeframe for completing a major rehabilitation project for both bridges. This would alleviate traffic concerns by limiting lane closures to occurring one bridge at a time.

Some aspects of a major rehabilitation will likely require complete bridge closure. For example, this would likely include projects such as replacement of interior gusset plates. While there may be certain projects which could be done concurrently on both bridges, development of a comprehensive construction schedule is outside the scope of this study.

The anticipated scope of a future major rehabilitation undertaking would include the following items:

- b.1 Truss Span Deck Replacement
- b.2 Stringer Replacement/Repair
- b.3 Floorbeam Repair
- b.4 Suspender Cable Replacement
- b.5 Replace Abutment Spans
- b.6 Bearing Repairs
- b.7 Joint Replacement
- b.8 Minor Steel Truss Repairs
- b.9 Major Steel Truss Repairs
- b.10 Paving (Overlay)
- b.11 Painting of Structural Steel

b.1 Truss Span Deck Replacement:

The lightweight concrete filled steel grid deck was replaced in 1979 on the Bourne Bridge and 1981 on the Sagamore Bridge. A typical service life for this type of deck is 40+ years. Although the current structural condition of the deck is good, it means that the deck will likely need replacing c. 2025. Replacement of the deck would require significant lane closures and would run concurrent with major steel repairs below the deck.

b.2 Stringer Replacement/Repair:

The current stringers are in overall fair condition for both bridges, except for the fascia stringers, some of which exhibit significant pitting and section loss to the bottom flanges. Numerous original stringers were replaced during the deck replacement projects for both bridges in 1979 and 1981. Although there are currently no structural load capacity issues with these stringers, in the event of a deck replacement project, these fascia stringers would be replaced to ensure structural load capacity is maintained. Therefore, fascia stringer replacement would take place in conjunction with a deck replacement project c.2025. Repairs to stringers would involve the addition of cover plates (to improve load carrying capacity) and the removal of any fatigue sensitive details.

b.3 Floorbeam Repair:

The floorbeams are in fair condition on both bridges. However, the floorbeams under the joints are particularly vulnerable to corrosion due to leaking of failed bridge joints. No floorbeams have been replaced on either bridge, but the recent steel repair project in 2012 included repairs to some floorbeams. It is likely that the number of required floorbeam repairs will increase as the bridge ages. Repairs would likely include the addition or replacement of cover plates and the removal of any fatigue sensitive details.

b.4 Suspender (Hanger) Cable Replacement:

The suspender, or hanger, cables were replaced in 1981 on the Sagamore Bridge and in 1986 on the Bourne Bridge. There are 13 pairs of cables per side of each bridge. Temporary jacking beams are required to remove cable pairs. This work can be done with the deck replacement project. Cables such as this typically have a service life about 50 years, but the service life varies based on the environment and loading experienced by the cables. Over time, degradation and elongation of the bridge cables will determine the need for replacement.

b.5 Replace Abutment Spans (Transverse Girders; T-Beams & Deck)

The concrete T-beams at the Bourne Bridge are in poor condition. The T-beams at the Sagamore Bridge are in fair condition. The T-beams were repaired c. 2000 at both bridges, but these repairs were localized. Further rehabilitation will require extensive concrete repairs to the beams to maintain their overall structural integrity. Complete replacement is required in order to significantly increase the service life of these elements. Though the present state of this damage does not adversely affect the structural adequacy, if neglected for extended periods of time structural adequacy would be affected.

Over one-third of the area of the concrete deck of the abutments at the Sagamore Bridge is in poor condition. The Bourne Bridge abutment concrete deck is on overall good condition. The concrete deck on the abutments has been repaired numerous times since original construction of the bridges. There are various repair materials making up the total depth of the concrete deck, much of which is deteriorated. This deteriorated condition results in premature failure of any pavement overlay. The decks require replacement to regain the overall integrity of the abutment spans.

Since the replacement of these elements will require the typical 3-phase traffic control approach, replacement of the abutment spans should be undertaken at the same time as a truss span deck replacement project.

b.6 Bearing Repairs:

The bearings are in overall poor condition at both bridges. There are 24 bearings at the Bourne and 8 at the Sagamore. Repairs would include any necessary seismic retrofits as well as installing new anchor bolts.

b.7 Joint Replacement:

The existing joints are in serious to good condition at both bridges. At the Sagamore Bridge, the modular joint system at the south abutment installed c. 1995 was in serious condition due to significant spalling of the concrete supports and deterioration of the support bars within the joint resulting in a vertical misalignment. This joint and supporting concrete was replaced in 2018. All of the compression seal joints were also replaced in 2018.

At the Bourne Bridge, the Pier 3 Waboflex deck joint exhibits significant deflection under live loads and general deterioration throughout. This joint was partially repaired in 2010. The Transflex modular joint at Pier 4 is also deteriorated, misaligned, and has broken splice keys. Minor temporary repairs were made c. 2005. The compression strip seals were all replaced in 2010 and on both bridges are now dislodged, missing, torn, or generally damaged and deteriorated. Both Pier 3, Pier 4, and all the compression joint seals were replaced in Spring of 2019.

b.8 Minor Steel Truss Repairs:

Minor steel repairs would include all aspects of the steel repair project completed in 2011-2013 on both bridges at a cost of $9.5 million. This included repairing or replacing some secondary bracings members, lacing bars, batten plates, etc., retrofitting main exterior truss gusset plates, and repairing various floorbeams and removing fatigue sensitive details on the trusses throughout the structure. Any further minor steel repairs would likely need to include further exterior gusset plate retrofits on the main truss members, as well as repairs to some of the main truss members, secondary bracing, floorbeams, and stringers.

b.9 Major Steel Truss Repairs:

Major steel repairs would include the replacement of various members, as needed. This would include complete replacement of floorbeams and interior gusset plates. This type of project requires extensive lane closures and most likely intermittent full bridge closure to all vehicular traffic during the course of this work.

Replacement of major supporting elements such as floorbeams would require complete closure during the replacement process. While this support system is in place, the bridge would likely have to be closed to all vehicular traffic. Removal of the deck at each floorbeam location to be replaced would likely be necessary.

Interior gusset plate replacement requires the temporary removal of numerous secondary bracing members and disconnecting the floorbeam and the main truss members from the gusset plate. These bridge members would require an extensive temporary support system. While this support system is in place, the bridge would likely have to be closed to all vehicular traffic.

b.10 Paving (Overlay):

While paving in itself is not a major rehabilitation item, it would be included as part of an overall Major Rehabilitation project. Paving was last accomplished in 2010 for both bridges. Paving would be done in conjunction with the deck replacement project.

A microsilica concrete overlay was put on the Sagamore Bridge abutment spans in 1987 and the Bourne abutment spans in 1988. This microsilica concrete was completely removed from the Sagamore Bridge abutment spans during the most recent paving in 2010 and was replaced with Rosphalt (Rosphalt is a proprietary product combining asphalt and a waterproofing substance into one layer). Only obviously deteriorated portions of the microsilica overlay were removed from the Bourne Bridge in 2010. The entire overlay was not removed.

b.11 Painting of Structural Steel:

While painting in itself is not a major rehabilitation item, it would be included as part of an overall Major Rehabilitation project. Painting of the bridges is the single best method for preserving the current condition of the structural steel. Active corrosion results in section loss and decreased load capacity of the members. Maintenance painting every 7 years would include spot cleaning and blasting to bare steel of significant coating deterioration/paint loss/corrosion and a brush-off blast of the entire structure prior to recoating the entire structure. Both bridges have undergone complete paint removal (deleading); the Bourne in 2006 and the Sagamore in 2014.

b.12 Traffic Management Issues During a Major Rehabilitation Project:

Traffic management will be a significant task during a Major Rehabilitation project. It will likely include multiple and lengthy lane closures throughout the duration of the project and significant time where complete bridge closure would be required. While this study did not analyze specific traffic control requirements or timeframes for the various major rehabilitation tasks, a generalized approach was used to provide an overall concept of what may be required for such a project.

The Major Rehab would likely require some form of lane closures and/or bridge closures. A Major Rehab project would include the following items that would likely require traffic control:

- Deck Replacement – This would include the replacement of fascia (exterior) stringers and various floorbeams and interior gusset plates which should be done concurrently with the removal of the deck. (Note, many steel replacement issues can and should be accomplished concurrently with the deck replacement).
- Exterior Gusset Plate Retrofit
- Interior Gusset Plate Repair or Replacement - This type of activity can be performed concurrently with the deck replacement.
- Suspender Cable Replacement
- Abutment Span Replacement (replacement of T-Beams and deck)
- Misc. Steel Repairs/Suicide Fence Repairs, etc.
- Misc. Concrete Repairs (abutment parapets, exterior of substructure, etc.)

- Paving
- Painting

The list above is not an all-encompassing list of items required during a Major Rehabilitation of either bridge, but rather an estimation of those activities which would likely require some form of traffic control utilizing NAE's typical 3-phase traffic control scheme, or which may require complete bridge closure. This is only meant to capture the larger critical issues. The engineering required to fully determine and document each specific major rehab item is outside the scope of this effort.

Table A-7-1 summarizes lane closure and full bridge closure timeframes for a Major Rehab of the Bourne and Sagamore Bridges. These are strictly gross estimates based on engineering judgement and similar previous work done at the bridges. The actual lane closure and bridge closure timeframes are best predicted as part of the development of engineering documents such as Plans and Specifications for each of these specific major rehabilitation activities.

Table A-7-1

MAJOR REHAB ACTIVITY	**BOURNE**	**SAGAMORE**
	LANE CLOSURE DURATION (DAYS)	
BRIDGE SUPERSTRUCTURE DECK REPLACEMENT (INCL. STRINGER REPLACEMENT); ABUTMENT SPAN REPLACEMENT; (CONCRETE T-BEAMS) MISC. STEEL REPAIRS, ETC.; EXTERIOR GUSSET PLATE RETROFITS; INTERIOR GUSSET PLATE REPAIRS; MISC. CONCRETE REPAIRS, ETC.	165	135
SUSPENDER CABLE REPLACEMENT	65	70
PAVING	30	25
PAINTING	220	150
TOTAL DAYS OF LANE CLOSURES	**480**	**380**
	FULL BRIDGE CLOSURE DURATION (DAYS)	
INTERIOR GUSSET PLATE REPLACEMENT	70	95
FLOORBEAM REPLACEMENT	110	35
TOTAL DAYS OF FULL BRIDGE CLOSURE	**180**	**130**

It was assumed that replacement of interior gusset plates and floorbeams would require complete bridge closure due to the nature of having to disconnect an entire truss panel point and temporarily support this with a saddle type beam located on the roadway deck above. Replacement of these interior gusset plates and floorbeams will require construction sequencing and methods that result in a full bridge closure to all vehicular traffic while these gusset plates and floorbeams are replaced.

It is anticipated that multiple bridge closure periods would be required over the course of a typical Major Rehabilitation project. Each closure would probably be sequenced and scheduled to have as minimal impacts as possible. Multiple interior gusset plate and/or floorbeam replacements could occur during any given period of full bridge closure, but time and physical constraints would still result in the likelihood of multiple bridge closures over the course of a Major Rehabilitation project.

Many of the rehabilitation items can be done concurrently with the total bridge superstructure deck replacement. For example, abutment span deck replacement, miscellaneous concrete repairs, miscellaneous steel repairs, and exterior and interior gusset plate repairs or retrofits could all be done concurrently with the bridge superstructure deck replacement.

The duration for lane and bridge closure would still be subjected to the time limits associated with NAE's normal O&M projects, which typically excludes placing traffic control on the bridge between Memorial Day through Columbus Day. Of course, weather delays, especially during the winter months, would extend the duration of any project. It is assumed that weather delays could account for 15-30 days during the winter months, based on past efforts.

The impacts on lengthy lane closures will be most significant for bridge superstructure deck replacement and replacement of the abutment spans (T-Beams and concrete deck). Time frames for items requiring full bridge closure will have enormous impacts on the local traffic pattern and likely the local economy, even if for just short lengths of time. Full bridge closures cannot be done piecemeal (i.e., 5 days during one month and 5 days the next month) but should be scheduled for specific lengths of time over the course of two or three construction seasons.

Critical path analysis of these types of rehab activities and required traffic control is outside the scope of this effort, but would be accomplished prior to, or during, the development of the Plans and Specifications for such a project.

b.13 Marine Traffic Management Issues During a Major Rehabilitation Project:

It was assumed that there would be minimal delays to marine navigation throughout the duration of a Major Rehabilitation project. Barge mounted cranes would likely not be necessary and were not used during the last major rehabilitation. Major Rehabilitation of both the Bourne and Sagamore Bridges was completed circa 1981. The work consisted of replacement of the bridge deck with a concrete-filled steel grid, replacement and repairs to deteriorated stringers, replacement of hanger cables, repair of secondary members, replacement of corroded rivets and lacing bars, and painting of the superstructure.

c. Bridge Replacement

This scenario postulates that two new vehicular bridges will be constructed, one parallel to the existing Bourne Bridge and the other parallel to the existing Sagamore Bridge. The existing Bourne and Sagamore Bridges would remain in service until the new bridges are constructed. For purposes of this study, a cable-stay bridge alternative was investigated. However, any bridge replacement would require further investigation to ascertain the most economical and favorable bridge type. This conceptual cable-stay bridge is based on the SR-1 bridge over the Chesapeake and Delaware Canal in Delaware.

This bridge type was chosen for this study, in part, because it is a USACE owned bridge over a marine navigation canal (the Chesapeake and Delaware Canal) of similar proportions to the Cape Cod Canal. It provides an alternative similar to what would be required for a new bridge to cross the Canal. A new bridge type and design have not been accomplished for this study. The bridge replacements described and shown below are only representative of what could be used as a replacement structure.

A new Bourne Bridge of this type would likely be approximately 19 to 23 spans with a total length of between about 3,500 to about 4,000 feet. The estimated length is based on the local topography, required elevation of the superstructure, accounting for sea level rise, and assuming a 4% roadway grade. It is also based on an arbitrary location of the abutments for each bridge.

This would be comprised precast segmental girders, cables for the cable-stay spans, and three spans of steel multi-girders. There would be two reinforced concrete abutments, 16 to 20 reinforced concrete piers, and two reinforced concrete pylons for the cable-stay span.

A new Sagamore Bridge of this type would likely be approximately 12 to 14 spans with a total length between about 2,400 to 3,000 feet. The length is based on the local topography, required elevation of the superstructure, accounting for sea level rise, and assuming a 4% roadway grade. It is also based on an arbitrary location of the abutments for each bridge.

This would be comprised of precast segmental girders, cables for the cable-stay spans, and three spans of steel multi-girders. There would be two reinforced concrete abutments, nine to 11 reinforced concrete piers, and two reinforced concrete pylons for the cable-stay span.

Conceptual bridge replacement profiles for both bridges are shown below. The final bridge alignment, height, grade, and overall configuration will likely be different from what is proposed for this study.

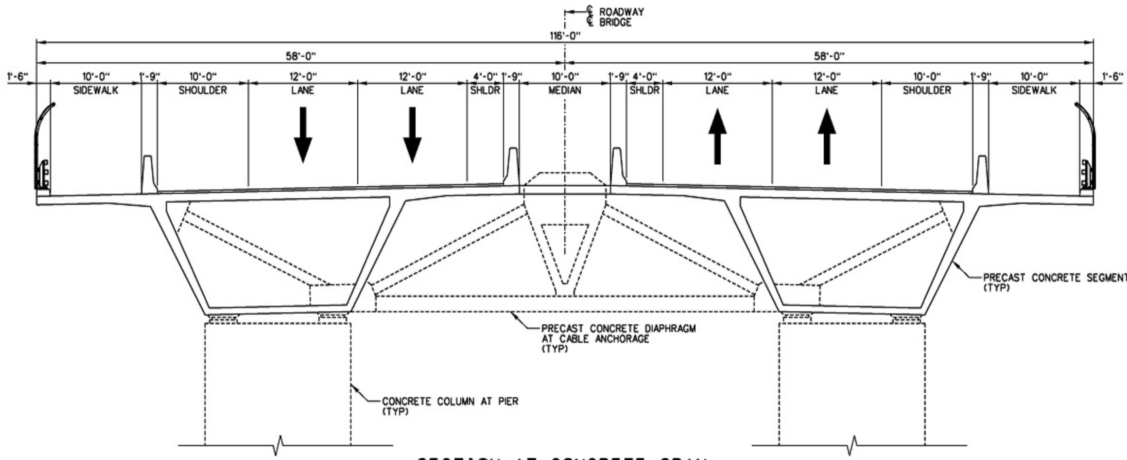
c.1 Traffic Management Issues During a Bridge Replacement Project:

Traffic management will be required during a Bridge Replacement project, but will not be as extensive and the time associated with lane closures will not be nearly as significant as for a Major Rehabilitation project. It will likely consist of various changes to traffic patterns necessitated by the reconfiguration of the approach roads to either bridge. It will not include any significant lane or bridge closures on the existing Bourne or Sagamore bridges themselves, whereas the new bridges would be constructed adjacent to the existing bridges.

This study did not analyze specific approach road traffic control requirements for the replacement of the Bourne and Sagamore bridges.

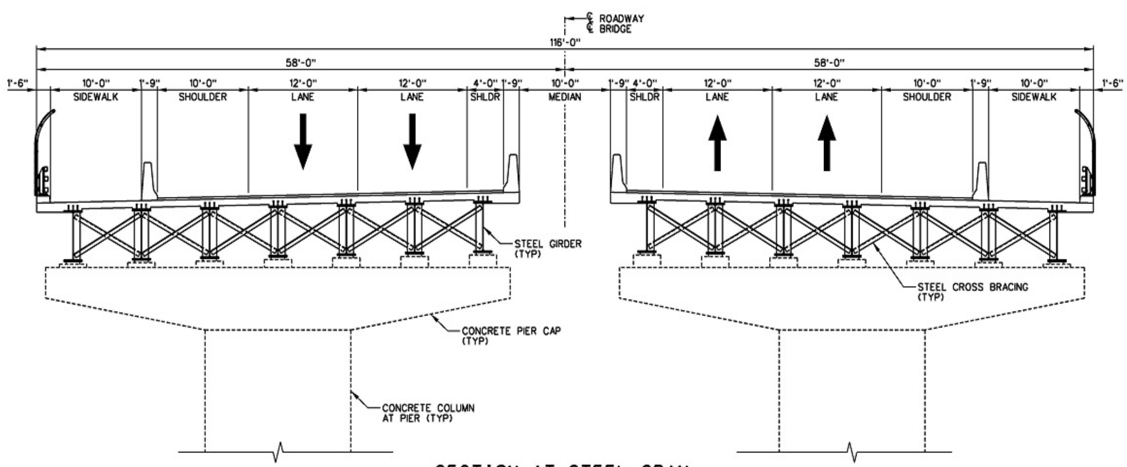
c.2 Marine Traffic Management Issues During a Bridge Replacement Project:

It was assumed that there will be at least 30 days where marine traffic will be delayed due to construction of a new bridge, and demolition of the existing bridge superstructure and water piers. Barge mounted cranes would likely be required for both demolition of the existing bridges and construction of the new bridges. The presence of these barges would lead to limiting marine navigation during various periods of construction. The actual number of days will need to be determined during future development of a Bridge Replacement project.



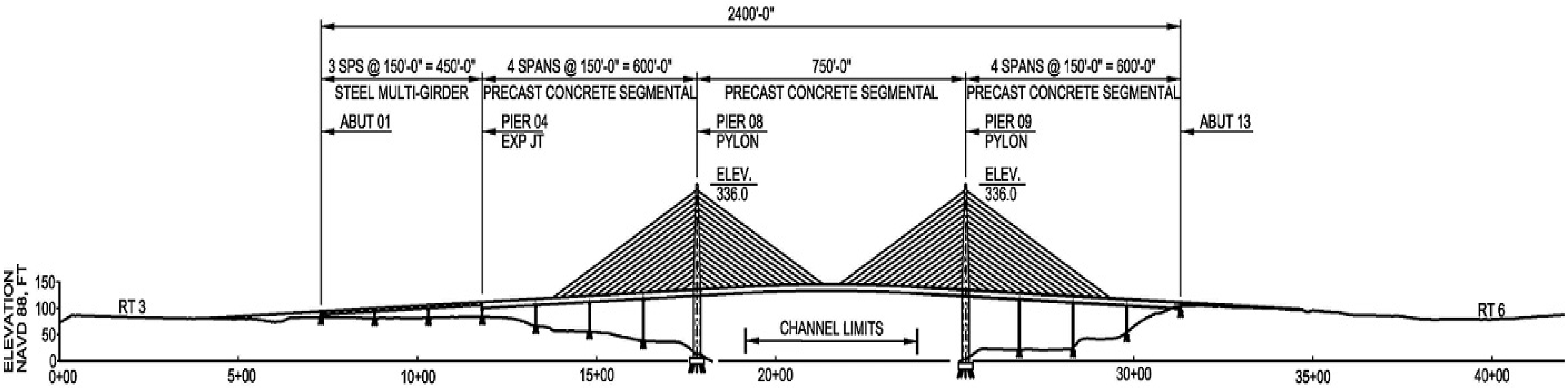
SECTION AT CONCRETE SPAN

SCALE: 3/16" = 1'-0"

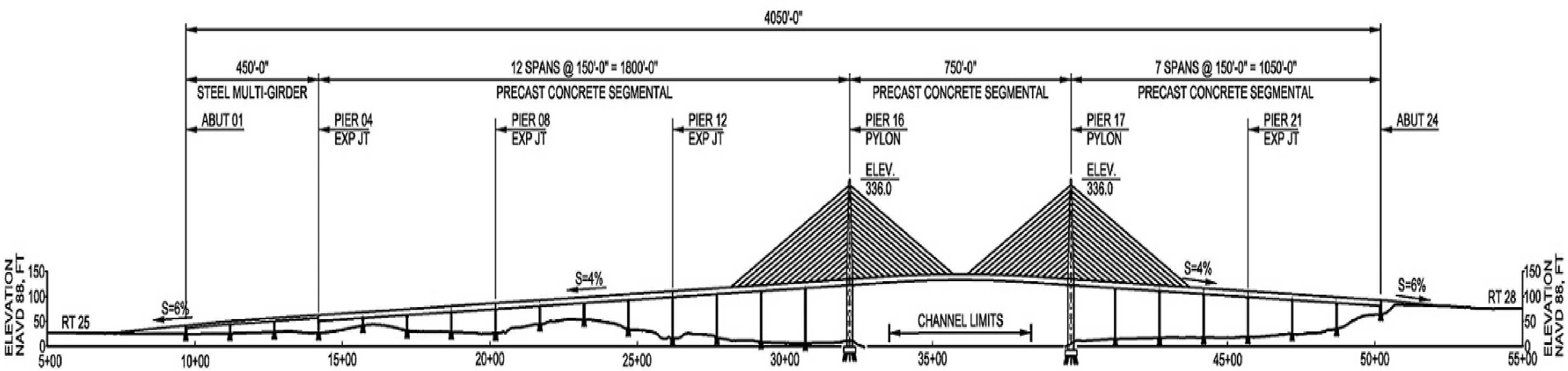


SECTION AT STEEL SPAN

SCALE: 3/16" = 1'-0"



SAGAMORE BRIDGE REPLACEMENT PROFILE



BOURNE BRIDGE REPLACEMENT PROFILE

8. STRUCTURAL RELIABILITY

a. Objective

This structural reliability analysis serves as the probabilistic basis for an economic analysis that drives the decision making process by demonstrating the best economic alternative for addressing the deteriorating performance of the ageing Bourne and Sagamore Bridges. This analysis was conducted in 2016, hence the reliability calculations are prepared for years 2016 to 2065, consistent with the prescribed 50-year service life for economic analysis. This period of analysis is suitable because the condition ratings for the Bourne and Sagamore bridges have not changed since 2014 for the Bourne Bridge and 2013 for the Sagamore Bridge. The condition ratings is what was used as the means of identifying a limit state of unsatisfactory performance.

b. Economic Alternatives

Three economic alternatives are identified for evaluation as follows:

(1) Base Condition. The Base Condition, synonymous with a "without project" condition, assumes that the bridges will continue to be operated efficiently and with due diligence for vehicular and marine safety. In the event of unsatisfactory performance of a bridge component, it is assumed that emergency funding will be made available to address the deficiency. This scenario portrays a condition where the reliability of the bridges is allowed to fall below the current condition, but that the bridge remains functional.

(2) Major Rehabilitation. This scenario assumes that all known structural deficiencies on both bridges will be addressed under a Major Rehabilitation Contract.

(3) Bridge Replacement. This scenario postulates that two new vehicular bridges will be constructed, one parallel to the existing Bourne Bridge and the other parallel to the existing Sagamore Bridge. The existing Bourne and Sagamore Bridges would remain in service until the new bridges are constructed.

c. Reliability Concepts

Reliability is defined as the probability that unsatisfactory performance will not occur. A "Limit State" is defined as the point at which unsatisfactory performance will occur or the engineering consequence will have some adverse economic impact. For this study, the limit state for unsatisfactory performance is defined as the physical condition where any of the bridges' critical elements is assigned a Condition Rating of 4 (Poor Condition) or less in accordance with protocols of the National Bridge Inspection Standard (NBIS).

Defining unsatisfactory performance based on the physical condition of the bridges using NBIS Condition Rating codes provides a viable way of determining a set of data points necessary for the regression analysis. USACE has historic data pertaining to the condition rating codes and this data can also be extrapolated for further analysis. In addition, this type of data is consistent with information in the national bridge inventory where data from similar types of bridges of similar age and environment can also be used for comparison purposes. The corrosion data reported in Section 6 above is ultimately related to the condition ratings through the performance of prior and subsequent Routine or In-Depth inspections of the bridges to assess the overall condition of the superstructure, deck, and substructure.

d. Deterioration Models

The overall reliability of the bridges is governed by three critical elements: superstructure, bridge deck, and substructure. Unsatisfactory performance of one or more of these critical elements would lead to unsatisfactory performance of the entire bridge. In order to assess the engineering reliability of the bridges, a probabilistic hazard function was developed for each of the three critical elements. For each critical element, a two-parameter (defined by a shape parameter and a scale parameter) Weibull Probability Distribution was developed to predict deteriorating bridge element performance over a fifty-year service life. The Weibull Probability Distribution is well accepted in academia and engineering literature as a methodology for assessing reliability and failure rates. Mathematical expressions for the Weibull Probability Distribution are as follows:

$F(t)$ = Cumulative Distribution Function = $1 – e^\wedge [(-t/\eta)^\wedge\beta]$

$h(t)$ = Annual Hazard (Failure) Rate = $(\beta/\eta)[(t/\eta)^\wedge\beta\text{-}1]$

$L(t)$ = Reliability Function = $1 – F(t)$

Where,

β = shape parameter

η = scale parameter

t = time

Calibration of the Weibull Probability Distribution for the superstructure and bridge deck was performed by regression analysis of data sets obtained from the National Bridge Inspection (NBI) database. The NBI database is the repository of information on all bridges in the nation and contains information on the year the bridge was built, the year the bridge was reconstructed, and summary condition ratings for the superstructure, bridge deck, and substructure.

For the superstructure and bridge deck, the NBI database was queried for bridges of similar construction and age to that of the Bourne and Sagamore Bridges located in New England, New York, and over the Chesapeake-Delaware Canal, which are geographic areas with similar environmental exposures. Where the NBI data indicated an entry for "Year Reconstructed," additional historical information was obtained by searching the internet for details of the reconstruction work. When reconstruction occurred, the bridges were generally verified or otherwise assumed to be in "Poor Condition." It is important to note that in the NBI database, an entry for "Year Reconstructed" is the date at which the rehabilitation work was completed. To calibrate the Weibull parameters, the following method was used to determine the time at which the bridge deteriorated to Poor Condition:

Time to Poor Condition = Year Reconstructed – Year Built – 5 Years

The five-year factor was adopted in the expression above as a reasonable estimate of the period it takes to program funding, perform design, and execute a remedial contract.

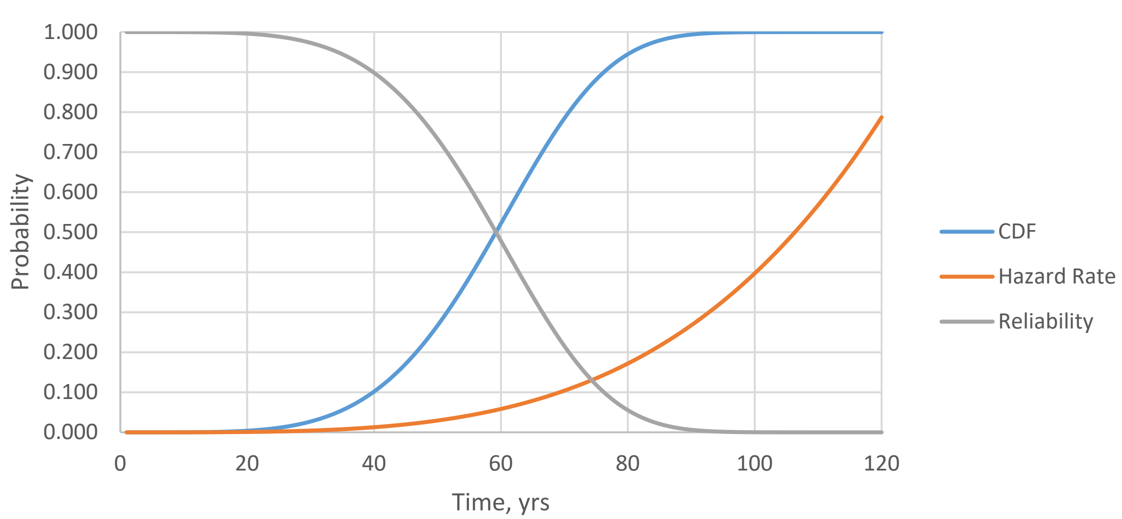
Data for substructure elements are not easily searchable in the NBI database. For this critical element, standard data points adopted by the U.S Army Corps of Engineers' Risk Management Center were used for the substructure deterioration model. These data points are contained in an electronic file titled "National Weibull Curve, Concrete A" and represent conglomerate data points for reinforced concrete locks, walls, and bridge piers.

The Weibull Distribution parameters used for each of the three critical elements, are summarized in the table below. The Weibull shape parameter, β, is also known as the Weibull slope. The value of β is equal to the slope of the line in a probability plot. If the Weibull scale parameter, η, is greater than 1, this indicates that the failure rate increases with time. This happens if there is an "aging" process, or parts that are more likely to fail as time goes on. These parameters were derived from the hazard function curves.

Weibull Cumulative Distribution Function (CDF) and hazard rates developed for superstructure, decks, and substructure are presented in Figures 8-1, 8-2, and 8-3, respectively. Weibull CDF is the probability of an event occurring within the time "t". The hazard rate is a conditional failure rate in relation to the reliability of a system or component. The hazard functions are presented in tables 8-1 through 8-9 at the end of this section.

WEIBULL DISTRIBUTION PARAMETERS		
Bridge Element	Shape Parameter (β)	Scale Parameter (η)
Superstructure	4.752	63.97
Bridge Deck	4.909	59.73
Substructure	4.000	156.00

Figure 8-1 - Superstructure - Weibull CDF and Hazard Rate



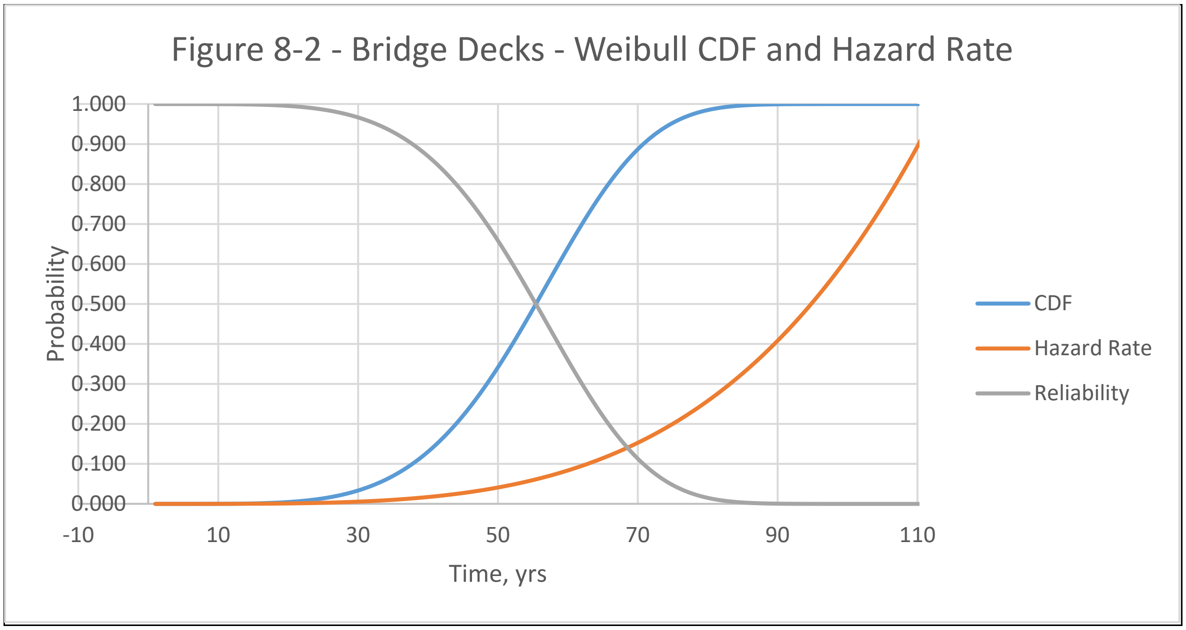


Figure 8-2 - Bridge Decks - Weibull CDF and Hazard Rate
1.000
0.900
0.800
0.700
0.600
0.500
0.400
0.300
0.200
0.100
0.000
Probability
-10
10
30
50
70
90
110
Time, yrs
CDF
Hazard Rate
Reliability

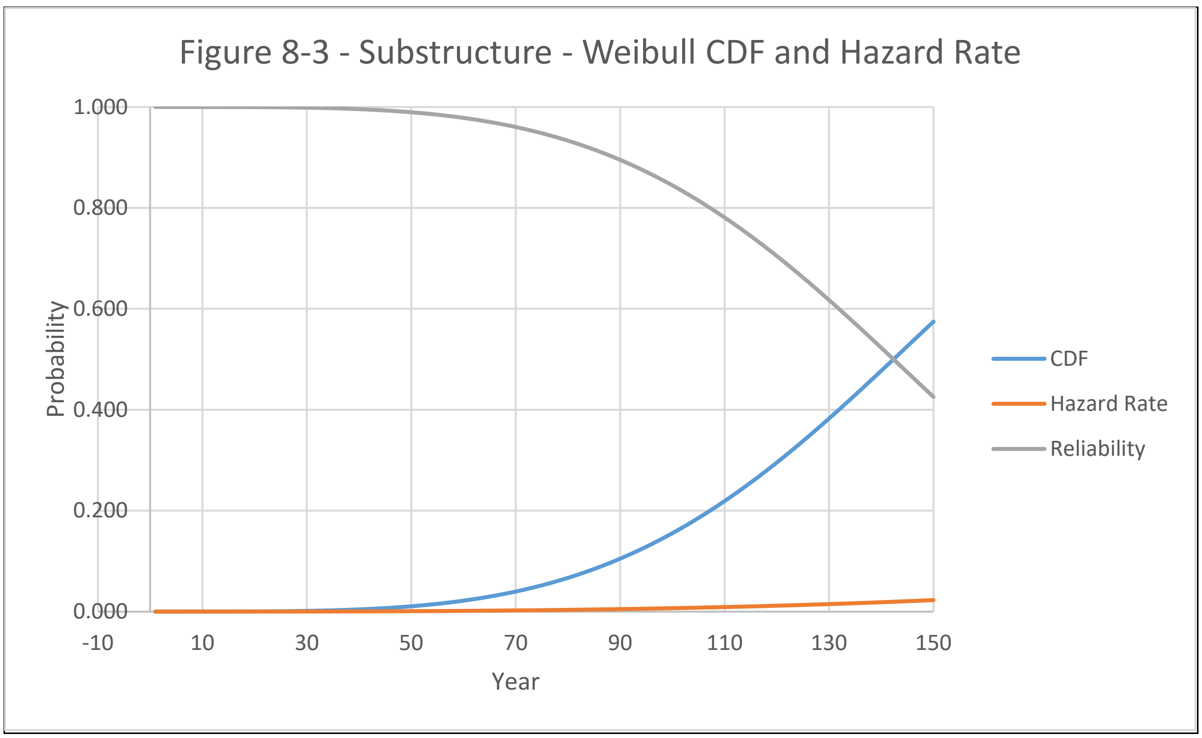


Figure 8-3 - Substructure - Weibull CDF and Hazard Rate
1.000
0.800
0.600
0.400
0.200
0.000
Probability
-10
10
30
50
70
90
110
130
150
Year
CDF
Hazard Rate
Reliability

e. Reliability Calculations for the Base Condition

Major Rehabilitation of both the Bourne and Sagamore Bridges was completed circa 1981. The work consisted of replacement of the bridge deck with a concrete-filled steel grid, replacement and repairs to deteriorated stringers, replacement of hanger cables, repair of secondary members, replacement of corroded rivets and lacing bars, and painting of the superstructure. For development of the deterioration model, it is assumed that the rehabilitation of the superstructure extended the service life by twenty years, i.e. the time variable is reset to twenty years prior for years 1981 and beyond for purposes of computing the superstructure's reliability. Since the bridge deck was replaced completely as part of the major rehabilitation, the time variable is reset to zero in year 1981. No adjustment of the time variable for the substructure was made since only routine maintenance consisting of crack sealing and spall repairs has been performed over the life of the bridges. See Tables A-8-1, A-8-2, A-8-3 for the predicted reliability under the Base Condition for the superstructure, bridge deck, and substructure, respectively.

f. Reliability Calculations for Major Rehabilitation and Bridge Replacement Alternatives

For the Major Rehabilitation economic alternative, a postulated major rehabilitation in year 2016 is assumed to extend the service life of the superstructure and substructure by an additional twenty years and reset the time variable for the bridge deck to zero at the beginning of calendar year 2016 (assuming that deck again would be replaced completely). See Tables A-8-4, A-8-5, and A-8-6 for the predicted reliability under the Major Rehabilitation alternative for the superstructure, bridge deck, and substructure, respectively.

For the Bridge Replacement economic alternative, the time variables for computing the reliability of the superstructure, bridge deck, and substructure are all reset to zero at the beginning of calendar year 2016. See Tables A-8-7, A-8-8, and A-8-9 for the predicted reliability under the Bridge Replacement alternative for the superstructure, bridge deck, and substructure, respectively.

g. Consequence of Unsatisfactory Performance

The consequences of unsatisfactory performance, defined for this study as an NBI Condition rating equal to, or less than, 4 (Poor Condition) for the superstructure, bridge deck, or substructure on either bridge, are presented on an Event Tree for each critical element under each economic alternative. In addition to the contract costs for repair work, the economic factors associated with unsatisfactory performance predominantly are the delays to vehicular traffic and commercial marine vessels navigating the Cape Cod Canal.

All repair work on the superstructure and bridge deck require vehicular lane closures to facilitate contractor activities. Typically, these lane closures restrict travel to one lane in each direction. Historically, temporary lane closures have been in effect for a minimum of approximately nine months during the course of repair contracts.

As previously stated, full closure of the bridge will be required for shorter time periods to allow critical replacement of certain bridge components, such as interior gusset plates and floorbeams. It is anticipated that multiple bridge closure periods would be required over the course of a typical Major Rehabilitation project and each closure would probably be for a period of about 2 weeks. Multiple interior gusset plate and/or floorbeam replacements could occur during any given period of full bridge closure, but time and physical constraints would still result in the likelihood of multiple bridge closures over the course of a Major Rehabilitation project.

Repairs to the substructure would require closure or delays to commercial marine vessels in additional to limited vehicular lane closures. For this reliability study, the substructure components are limited to the bridge piers located in the waterway. The abutments for both bridges can accessed from land-based construction methods and would not impact marine vessels.

An Event Trees for each of the critical elements has been developed to portray the full range of consequences caused by incidents ranging from localized structural defects to the remote probability of catastrophic damage. Given that the Bourne and Sagamore Bridges were both opened to traffic in 1935, exposed to similar environmental and load conditions, and maintained at approximately the same intervals, the Event Trees account for the probability that a structural defect manifested on one bridge will dictate that similar repair work be performed on the sister bridge. See Figures A-8-4, A-8-5, and A-8-6 for the Event Trees for the superstructure, bridge deck, and substructure, respectively.

h. Results of the Reliability Analysis

The cumulative reliability of the superstructure, bridge deck, and substructure for each of the alternatives at the end of the 50-year period (2016-2065) for economic evaluation is summarized in the table below. These cumulative reliabilities are used as a basis of comparison of the three alternatives, not as the basis for initiating any specific project or type of work.

CUMULATIVE RELIABILITY IN 2065			
Bridge Element	Base Condition	Major Rehabilitation	Bridge Replacement
Superstructure	0.000	0.006	0.733
Bridge Deck	0.005	0.659	0.659
Substructure	0.617	0.781	0.990

A review of the predicted reliabilities in this table indicate that the Base Condition alternative yields the lowest reliability for all three of the critical elements. The deterioration model predicts with near certainty that both superstructure and bridge deck will be performing unsatisfactorily at the end of the 50-year period of evaluation.

The Major Rehabilitation alternative predicts reliabilities that are an improvement over the Base Condition, but the superstructure reliability of 0.006 makes this alternative a poor investment option.

The Bridge Replacement alternative offers the highest reliability of the three economic alternatives under consideration. This alternative will allow the U.S Army Corps of Engineers to fulfill its responsibility to provide continued safe passage for vehicular and marine traffic.

j. Structural Conclusion.

This engineering reliability analysis shows that the Bridge Replacement alternative offers the highest reliability of these three alternatives.

Providing a replacement for the existing spans in-kind with respect to the number of through traffic lanes would not conform to current design guidance for bridges and highways. For this reason, providing new bridges without auxiliary lanes would not be consistent with best practices for traffic safety, and any plan for a 4-lane bridge will not be carried forward into a detailed analysis.

The two existing bridges with their four through traffic lanes were designed and built in the 1930s to serve far lower traffic volumes than those served by the bridges today. Modern highway design guidance, including AASHTO highway and bridge design specifications and MassDOT design guidance require including auxiliary lanes for entering and exiting traffic to transition into or out of through traffic safely.

Furthermore, qualitatively, a Major Rehabilitation project will have significant socio-economic impacts on the surrounding region due to long-term ongoing traffic delays and disruptions. The Base Condition, or without project, alternative, would likely create troublesome situations during the 50-year study period. For example, at some point the bridges will need to have weight restrictions, and likely undergo emergency structural repairs, which may or may not have an impact on load postings. Unplanned or emergency maintenance is costly and could lead to significant traffic issues at uncertain times of the year, possibly even during the busiest summer months for tourism. This would have a significant economic impact on the region's businesses.

Despite their testament to engineering accomplishments of the 19th Century and aesthetic charm, these 80 year old steel bridges are beyond their functional service life. While both of these bridges can be maintained to prolong their overall structural integrity, both are already functionally obsolete. In addition, both bridges will likely need load postings and truck traffic restrictions at some point due to corrosion and deterioration causing a decrease of structural capacity of various steel members, even if the bridges are maintained in a state of good repair. The Bourne and Sagamore bridges are not suitable for continued operation as a primary link in the highway system of southeastern Massachusetts and Cape Cod.

TABLE A-8-1: HAZARD FUNCTION FOR SUPERSTRUCTURE-BASE CONDITION

Years 2016 to 2065

Weibull Distribution:
Shape Parameter= 4.752
Scale Parameter= 63.97

Where,
h(t) = Failure Rate Function
F(t) = Cumulative Distribution Function
L(t) = Reliability Function
= 1 - F(t)

YEAR	Time *	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2016	61	0.0621	0.5497	0.4503
2017	62	0.0661	0.5776	0.4224
2018	63	0.0701	0.6054	0.3946
2019	64	0.0744	0.6329	0.3671
2020	65	0.0789	0.6600	0.3400
2021	66	0.0835	0.6865	0.3135
2022	67	0.0884	0.7123	0.2877
2023	68	0.0934	0.7373	0.2627
2024	69	0.0987	0.7614	0.2386
2025	70	0.1042	0.7844	0.2156
2026	71	0.1098	0.8063	0.1937
2027	72	0.1158	0.8269	0.1731
2028	73	0.1219	0.8463	0.1537
2029	74	0.1283	0.8644	0.1356
2030	75	0.1349	0.8811	0.1189
2031	76	0.1418	0.8965	0.1035
2032	77	0.1489	0.9105	0.0895
2033	78	0.1563	0.9231	0.0769
2034	79	0.1640	0.9345	0.0655
2035	80	0.1719	0.9446	0.0554
2036	81	0.1801	0.9536	0.0464
2037	82	0.1886	0.9614	0.0386
2038	83	0.1974	0.9682	0.0318
2039	84	0.2064	0.9740	0.0260
2040	85	0.2158	0.9789	0.0211

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2041	86	0.2255	0.9831	0.0169
2042	87	0.2355	0.9866	0.0134
2043	88	0.2458	0.9895	0.0105
2044	89	0.2564	0.9918	0.0082
2045	90	0.2674	0.9937	0.0063
2046	91	0.2787	0.9952	0.0048
2047	92	0.2904	0.9964	0.0036
2048	93	0.3024	0.9973	0.0027
2049	94	0.3148	0.9980	0.0020
2050	95	0.3276	0.9986	0.0014
2051	96	0.3407	0.9990	0.0010
2052	97	0.3542	0.9993	0.0007
2053	98	0.3681	0.9995	0.0005
2054	99	0.3824	0.9997	0.0003
2055	100	0.3971	0.9998	0.0002
2056	101	0.4122	0.9998	0.0002
2057	102	0.4277	0.9999	0.0001
2058	103	0.4437	0.9999	0.0001
2059	104	0.4600	1.0000	0.0000
2060	105	0.4768	1.0000	0.0000
2061	106	0.4941	1.0000	0.0000
2062	107	0.5118	1.0000	0.0000
2063	108	0.5300	1.0000	0.0000
2064	109	0.5487	1.0000	0.0000
2065	110	0.5678	1.0000	0.0000

* Note: The bridges were constructed in 1935. Major Rehabilitation of the superstructures was performed in 1981. This deterioration model assumes that the Major Rehabilitation extended the service life by 20 years.

TABLE A-8-2: HAZARD FUNCTION FOR BRIDGE DECK-BASE CONDITION

Years 2016 to 2065

Weibull Distribution:
Shape Parameter= 4.909
Scale Parameter= 59.73

Where,
h(t) = Failure Rate Function
F(t) = Cumulative Distribution Function
L(t) = Reliability Function
= 1 - F(t)

YEAR	Time *	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2016	35	0.0102	0.0700	0.9300
2017	36	0.0114	0.0799	0.9201
2018	37	0.0126	0.0909	0.9091
2019	38	0.0140	0.1029	0.8971
2020	39	0.0155	0.1161	0.8839
2021	40	0.0171	0.1304	0.8696
2022	41	0.0189	0.1459	0.8541
2023	42	0.0207	0.1626	0.8374
2024	43	0.0227	0.1806	0.8194
2025	44	0.0249	0.1999	0.8001
2026	45	0.0272	0.2205	0.7795
2027	46	0.0296	0.2423	0.7577
2028	47	0.0322	0.2653	0.7347
2029	48	0.0350	0.2896	0.7104
2030	49	0.0379	0.3150	0.6850
2031	50	0.0410	0.3415	0.6585
2032	51	0.0443	0.3690	0.6310
2033	52	0.0478	0.3974	0.6026
2034	53	0.0515	0.4266	0.5734
2035	54	0.0554	0.4564	0.5436
2036	55	0.0595	0.4867	0.5133
2037	56	0.0639	0.5174	0.4826
2038	57	0.0685	0.5483	0.4517
2039	58	0.0733	0.5792	0.4208
2040	59	0.0783	0.6099	0.3901

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2041	60	0.0836	0.6403	0.3597
2042	61	0.0892	0.6700	0.3300
2043	62	0.0951	0.6991	0.3009
2044	63	0.1012	0.7272	0.2728
2045	64	0.1077	0.7543	0.2457
2046	65	0.1144	0.7801	0.2199
2047	66	0.1214	0.8045	0.1955
2048	67	0.1288	0.8275	0.1725
2049	68	0.1364	0.8489	0.1511
2050	69	0.1445	0.8687	0.1313
2051	70	0.1528	0.8868	0.1132
2052	71	0.1615	0.9033	0.0967
2053	72	0.1706	0.9181	0.0819
2054	73	0.1800	0.9313	0.0687
2055	74	0.1899	0.9429	0.0571
2056	75	0.2001	0.9530	0.0470
2057	76	0.2107	0.9617	0.0383
2058	77	0.2218	0.9692	0.0308
2059	78	0.2333	0.9754	0.0246
2060	79	0.2452	0.9807	0.0193
2061	80	0.2575	0.9850	0.0150
2062	81	0.2704	0.9884	0.0116
2063	82	0.2836	0.9912	0.0088
2064	83	0.2974	0.9935	0.0065
2065	84	0.3117	0.9952	0.0048

* Note: Bridge decks were replaced during the 1981 Major Rehabilitation.

TABLE A-8-3: HAZARD FUNCTION FOR SUBSTRUCTURE-BASE CONDITION

Years 2016 to 2065

Weibull Distribution:
Shape Parameter= 4
Scale Parameter= 156

Where,
h(t) = Failure Rate Function
F(t) = Cumulative Distribution Function
L(t) = Reliability Function
= 1 - F(t)

YEAR	Time *	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2016	81	0.0036	0.0701	0.9299
2017	82	0.0037	0.0735	0.9265
2018	83	0.0039	0.0770	0.9230
2019	84	0.0040	0.0806	0.9194
2020	85	0.0041	0.0844	0.9156
2021	86	0.0043	0.0882	0.9118
2022	87	0.0044	0.0922	0.9078
2023	88	0.0046	0.0963	0.9037
2024	89	0.0048	0.1005	0.8995
2025	90	0.0049	0.1049	0.8951
2026	91	0.0051	0.1093	0.8907
2027	92	0.0053	0.1139	0.8861
2028	93	0.0054	0.1187	0.8813
2029	94	0.0056	0.1235	0.8765
2030	95	0.0058	0.1285	0.8715
2031	96	0.0060	0.1336	0.8664
2032	97	0.0062	0.1388	0.8612
2033	98	0.0064	0.1442	0.8558
2034	99	0.0066	0.1497	0.8503
2035	100	0.0068	0.1554	0.8446
2036	101	0.0070	0.1611	0.8389
2037	102	0.0072	0.1670	0.8330
2038	103	0.0074	0.1731	0.8269
2039	104	0.0076	0.1792	0.8208
2040	105	0.0078	0.1855	0.8145

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2041	106	0.0080	0.1920	0.8080
2042	107	0.0083	0.1985	0.8015
2043	108	0.0085	0.2052	0.7948
2044	109	0.0087	0.2121	0.7879
2045	110	0.0090	0.2190	0.7810
2046	111	0.0092	0.2261	0.7739
2047	112	0.0095	0.2333	0.7667
2048	113	0.0097	0.2407	0.7593
2049	114	0.0100	0.2481	0.7519
2050	115	0.0103	0.2557	0.7443
2051	116	0.0105	0.2634	0.7366
2052	117	0.0108	0.2712	0.7288
2053	118	0.0111	0.2792	0.7208
2054	119	0.0114	0.2872	0.7128
2055	120	0.0117	0.2954	0.7046
2056	121	0.0120	0.3037	0.6963
2057	122	0.0123	0.3121	0.6879
2058	123	0.0126	0.3206	0.6794
2059	124	0.0129	0.3291	0.6709
2060	125	0.0132	0.3378	0.6622
2061	126	0.0135	0.3466	0.6534
2062	127	0.0138	0.3555	0.6445
2063	128	0.0142	0.3644	0.6356
2064	129	0.0145	0.3735	0.6265
2065	130	0.0148	0.3826	0.6174

* Note: The bridges were constructed in 1935. Maintenance of the reinforced concrete substructures has been performed on an as-needed basis.
Repair work consisted of crack sealing and spall repairs.

TABLE A-8-4: HAZARD FUNCTION FOR SUPERSTRUCTURE-MAJOR REHABILITATION

Years 2016 to 2065

Weibull Distribution:
Shape Parameter= 4.752
Scale Parameter= 63.97

Where,
h(t) = Failure Rate Function
F(t) = Cumulative Distribution Function
L(t) = Reliability Function
= 1 - F(t)

YEAR	Time *	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2016	41	0.0140	0.1138	0.8862
2017	42	0.0153	0.1267	0.8733
2018	43	0.0167	0.1405	0.8595
2019	44	0.0182	0.1554	0.8446
2020	45	0.0198	0.1714	0.8286
2021	46	0.0216	0.1883	0.8117
2022	47	0.0234	0.2063	0.7937
2023	48	0.0253	0.2254	0.7746
2024	49	0.0273	0.2455	0.7545
2025	50	0.0295	0.2666	0.7334
2026	51	0.0317	0.2887	0.7113
2027	52	0.0341	0.3118	0.6882
2028	53	0.0367	0.3357	0.6643
2029	54	0.0393	0.3605	0.6395
2030	55	0.0421	0.3860	0.6140
2031	56	0.0451	0.4122	0.5878
2032	57	0.0482	0.4390	0.5610
2033	58	0.0514	0.4662	0.5338
2034	59	0.0548	0.4938	0.5062
2035	60	0.0584	0.5217	0.4783
2036	61	0.0621	0.5497	0.4503
2037	62	0.0661	0.5776	0.4224
2038	63	0.0701	0.6054	0.3946
2039	64	0.0744	0.6329	0.3671
2040	65	0.0789	0.6600	0.3400

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2041	66	0.0835	0.6865	0.3135
2042	67	0.0884	0.7123	0.2877
2043	68	0.0934	0.7373	0.2627
2044	69	0.0987	0.7614	0.2386
2045	70	0.1042	0.7844	0.2156
2046	71	0.1098	0.8063	0.1937
2047	72	0.1158	0.8269	0.1731
2048	73	0.1219	0.8463	0.1537
2049	74	0.1283	0.8644	0.1356
2050	75	0.1349	0.8811	0.1189
2051	76	0.1418	0.8965	0.1035
2052	77	0.1489	0.9105	0.0895
2053	78	0.1563	0.9231	0.0769
2054	79	0.1640	0.9345	0.0655
2055	80	0.1719	0.9446	0.0554
2056	81	0.1801	0.9536	0.0464
2057	82	0.1886	0.9614	0.0386
2058	83	0.1974	0.9682	0.0318
2059	84	0.2064	0.9740	0.0260
2060	85	0.2158	0.9789	0.0211
2061	86	0.2255	0.9831	0.0169
2062	87	0.2355	0.9866	0.0134
2063	88	0.2458	0.9895	0.0105
2064	89	0.2564	0.9918	0.0082
2065	90	0.2674	0.9937	0.0063

* Note: The bridges were constructed in 1935. Major Rehabilitation of the superstructures was performed in 1981. This deterioration model assumes that the 1981 Major Rehabilitation extended the service life by 20 years and a postulated 2016 Major Rehabilitation would extend the service life by an additional 20 years.

TABLE A-8-5: HAZARD FUNCTION FOR BRIDGE DECK-MAJOR REHABILITATION OR BRIDGE REPLACEMENT

Years 2016 to 2065

Weibull Distribution:
Shape Parameter= 4.909
Scale Parameter= 59.73

Where,
h(t) = Failure Rate Function
F(t) = Cumulative Distribution Function
L(t) = Reliability Function
= 1 - F(t)

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2016	1	0.0000	0.0000	1.0000
2017	2	0.0000	0.0000	1.0000
2018	3	0.0000	0.0000	1.0000
2019	4	0.0000	0.0000	1.0000
2020	5	0.0000	0.0000	1.0000
2021	6	0.0000	0.0000	1.0000
2022	7	0.0000	0.0000	1.0000
2023	8	0.0000	0.0001	0.9999
2024	9	0.0001	0.0001	0.9999
2025	10	0.0001	0.0002	0.9998
2026	11	0.0001	0.0002	0.9998
2027	12	0.0002	0.0004	0.9996
2028	13	0.0002	0.0006	0.9994
2029	14	0.0003	0.0008	0.9992
2030	15	0.0004	0.0011	0.9989
2031	16	0.0005	0.0016	0.9984
2032	17	0.0006	0.0021	0.9979
2033	18	0.0008	0.0028	0.9972
2034	19	0.0009	0.0036	0.9964
2035	20	0.0011	0.0046	0.9954
2036	21	0.0014	0.0059	0.9941
2037	22	0.0017	0.0074	0.9926
2038	23	0.0020	0.0092	0.9908
2039	24	0.0023	0.0113	0.9887
2040	25	0.0027	0.0138	0.9862

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2041	26	0.0032	0.0167	0.9833
2042	27	0.0037	0.0201	0.9799
2043	28	0.0043	0.0240	0.9760
2044	29	0.0049	0.0284	0.9716
2045	30	0.0056	0.0335	0.9665
2046	31	0.0063	0.0392	0.9608
2047	32	0.0072	0.0456	0.9544
2048	33	0.0081	0.0529	0.9471
2049	34	0.0091	0.0610	0.9390
2050	35	0.0102	0.0700	0.9300
2051	36	0.0114	0.0799	0.9201
2052	37	0.0126	0.0909	0.9091
2053	38	0.0140	0.1029	0.8971
2054	39	0.0155	0.1161	0.8839
2055	40	0.0171	0.1304	0.8696
2056	41	0.0189	0.1459	0.8541
2057	42	0.0207	0.1626	0.8374
2058	43	0.0227	0.1806	0.8194
2059	44	0.0249	0.1999	0.8001
2060	45	0.0272	0.2205	0.7795
2061	46	0.0296	0.2423	0.7577
2062	47	0.0322	0.2653	0.7347
2063	48	0.0350	0.2896	0.7104
2064	49	0.0379	0.3150	0.6850
2065	50	0.0410	0.3415	0.6585

TABLE A-8-6: HAZARD FUNCTION FOR SUBSTRUCTURE-MAJOR REHABILITATION

Years 2016 to 2065

Weibull Distribution:
Shape Parameter= 4
Scale Parameter= 156

Where,
h(t) = Failure Rate Function
F(t) = Cumulative Distribution Function
L(t) = Reliability Function
= 1 - F(t)

YEAR	Time *	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2016	61	0.0015	0.0231	0.9769
2017	62	0.0016	0.0246	0.9754
2018	63	0.0017	0.0262	0.9738
2019	64	0.0018	0.0279	0.9721
2020	65	0.0019	0.0297	0.9703
2021	66	0.0019	0.0315	0.9685
2022	67	0.0020	0.0335	0.9665
2023	68	0.0021	0.0355	0.9645
2024	69	0.0022	0.0376	0.9624
2025	70	0.0023	0.0397	0.9603
2026	71	0.0024	0.0420	0.9580
2027	72	0.0025	0.0444	0.9556
2028	73	0.0026	0.0468	0.9532
2029	74	0.0027	0.0494	0.9506
2030	75	0.0028	0.0520	0.9480
2031	76	0.0030	0.0548	0.9452
2032	77	0.0031	0.0576	0.9424
2033	78	0.0032	0.0606	0.9394
2034	79	0.0033	0.0637	0.9363
2035	80	0.0035	0.0668	0.9332
2036	81	0.0036	0.0701	0.9299
2037	82	0.0037	0.0735	0.9265
2038	83	0.0039	0.0770	0.9230
2039	84	0.0040	0.0806	0.9194
2040	85	0.0041	0.0844	0.9156

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2041	86	0.0043	0.0882	0.9118
2042	87	0.0044	0.0922	0.9078
2043	88	0.0046	0.0963	0.9037
2044	89	0.0048	0.1005	0.8995
2045	90	0.0049	0.1049	0.8951
2046	91	0.0051	0.1093	0.8907
2047	92	0.0053	0.1139	0.8861
2048	93	0.0054	0.1187	0.8813
2049	94	0.0056	0.1235	0.8765
2050	95	0.0058	0.1285	0.8715
2051	96	0.0060	0.1336	0.8664
2052	97	0.0062	0.1388	0.8612
2053	98	0.0064	0.1442	0.8558
2054	99	0.0066	0.1497	0.8503
2055	100	0.0068	0.1554	0.8446
2056	101	0.0070	0.1611	0.8389
2057	102	0.0072	0.1670	0.8330
2058	103	0.0074	0.1731	0.8269
2059	104	0.0076	0.1792	0.8208
2060	105	0.0078	0.1855	0.8145
2061	106	0.0080	0.1920	0.8080
2062	107	0.0083	0.1985	0.8015
2063	108	0.0085	0.2052	0.7948
2064	109	0.0087	0.2121	0.7879
2065	110	0.0090	0.2190	0.7810

* Note: The bridges were built in 1935 and maintenance of the reinforced concrete substructure has been performed as needed.
This deterioration model assumes that a Major Rehabilitation would extent the service life of the substructure by 20 years.

TABLE A-8-7: HAZARD FUNCTION FOR SUPERSTRUCTURE-BRIDGE REPLACEMENT

Years 2016 to 2065

Weibull Distribution:
Shape Parameter= 4.752
Scale Parameter= 63.97

Where,
h(t) = Failure Rate Function
F(t) = Cumulative Distribution Function
L(t) = Reliability Function
= 1 - F(t)

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2016	1	0.0000	0.0000	1.0000
2017	2	0.0000	0.0000	1.0000
2018	3	0.0000	0.0000	1.0000
2019	4	0.0000	0.0000	1.0000
2020	5	0.0000	0.0000	1.0000
2021	6	0.0000	0.0000	1.0000
2022	7	0.0000	0.0000	1.0000
2023	8	0.0000	0.0001	0.9999
2024	9	0.0000	0.0001	0.9999
2025	10	0.0001	0.0001	0.9999
2026	11	0.0001	0.0002	0.9998
2027	12	0.0001	0.0004	0.9996
2028	13	0.0002	0.0005	0.9995
2029	14	0.0002	0.0007	0.9993
2030	15	0.0003	0.0010	0.9990
2031	16	0.0004	0.0014	0.9986
2032	17	0.0005	0.0018	0.9982
2033	18	0.0006	0.0024	0.9976
2034	19	0.0008	0.0031	0.9969
2035	20	0.0009	0.0040	0.9960
2036	21	0.0011	0.0050	0.9950
2037	22	0.0014	0.0062	0.9938
2038	23	0.0016	0.0077	0.9923
2039	24	0.0019	0.0094	0.9906
2040	25	0.0022	0.0114	0.9886

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2041	26	0.0025	0.0138	0.9862
2042	27	0.0029	0.0165	0.9835
2043	28	0.0033	0.0195	0.9805
2044	29	0.0038	0.0230	0.9770
2045	30	0.0043	0.0270	0.9730
2046	31	0.0049	0.0315	0.9685
2047	32	0.0055	0.0365	0.9635
2048	33	0.0062	0.0421	0.9579
2049	34	0.0069	0.0484	0.9516
2050	35	0.0077	0.0553	0.9447
2051	36	0.0086	0.0630	0.9370
2052	37	0.0095	0.0715	0.9285
2053	38	0.0105	0.0807	0.9193
2054	39	0.0116	0.0908	0.9092
2055	40	0.0128	0.1018	0.8982
2056	41	0.0140	0.1138	0.8862
2057	42	0.0153	0.1267	0.8733
2058	43	0.0167	0.1405	0.8595
2059	44	0.0182	0.1554	0.8446
2060	45	0.0198	0.1714	0.8286
2061	46	0.0216	0.1883	0.8117
2062	47	0.0234	0.2063	0.7937
2063	48	0.0253	0.2254	0.7746
2064	49	0.0273	0.2455	0.7545
2065	50	0.0295	0.2666	0.7334

TABLE A-8-8: HAZARD FUNCTION FOR BRIDGE DECK-MAJOR REHABILITATION OR BRIDGE REPLACEMENT

Years 2016 to 2065

Weibull Distribution:
Shape Parameter= 4.909
Scale Parameter= 59.73

Where,
h(t) = Failure Rate Function
F(t) = Cumulative Distribution Function
L(t) = Reliability Function
= 1 - F(t)

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2016	1	0.0000	0.0000	1.0000
2017	2	0.0000	0.0000	1.0000
2018	3	0.0000	0.0000	1.0000
2019	4	0.0000	0.0000	1.0000
2020	5	0.0000	0.0000	1.0000
2021	6	0.0000	0.0000	1.0000
2022	7	0.0000	0.0000	1.0000
2023	8	0.0000	0.0001	0.9999
2024	9	0.0001	0.0001	0.9999
2025	10	0.0001	0.0002	0.9998
2026	11	0.0001	0.0002	0.9998
2027	12	0.0002	0.0004	0.9996
2028	13	0.0002	0.0006	0.9994
2029	14	0.0003	0.0008	0.9992
2030	15	0.0004	0.0011	0.9989
2031	16	0.0005	0.0016	0.9984
2032	17	0.0006	0.0021	0.9979
2033	18	0.0008	0.0028	0.9972
2034	19	0.0009	0.0036	0.9964
2035	20	0.0011	0.0046	0.9954
2036	21	0.0014	0.0059	0.9941
2037	22	0.0017	0.0074	0.9926
2038	23	0.0020	0.0092	0.9908
2039	24	0.0023	0.0113	0.9887
2040	25	0.0027	0.0138	0.9862

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2041	26	0.0032	0.0167	0.9833
2042	27	0.0037	0.0201	0.9799
2043	28	0.0043	0.0240	0.9760
2044	29	0.0049	0.0284	0.9716
2045	30	0.0056	0.0335	0.9665
2046	31	0.0063	0.0392	0.9608
2047	32	0.0072	0.0456	0.9544
2048	33	0.0081	0.0529	0.9471
2049	34	0.0091	0.0610	0.9390
2050	35	0.0102	0.0700	0.9300
2051	36	0.0114	0.0799	0.9201
2052	37	0.0126	0.0909	0.9091
2053	38	0.0140	0.1029	0.8971
2054	39	0.0155	0.1161	0.8839
2055	40	0.0171	0.1304	0.8696
2056	41	0.0189	0.1459	0.8541
2057	42	0.0207	0.1626	0.8374
2058	43	0.0227	0.1806	0.8194
2059	44	0.0249	0.1999	0.8001
2060	45	0.0272	0.2205	0.7795
2061	46	0.0296	0.2423	0.7577
2062	47	0.0322	0.2653	0.7347
2063	48	0.0350	0.2896	0.7104
2064	49	0.0379	0.3150	0.6850
2065	50	0.0410	0.3415	0.6585

TABLE A-8-9: HAZARD FUNCTION FOR SUBSTRUCTURE-BRIDGE REPLACEMENT

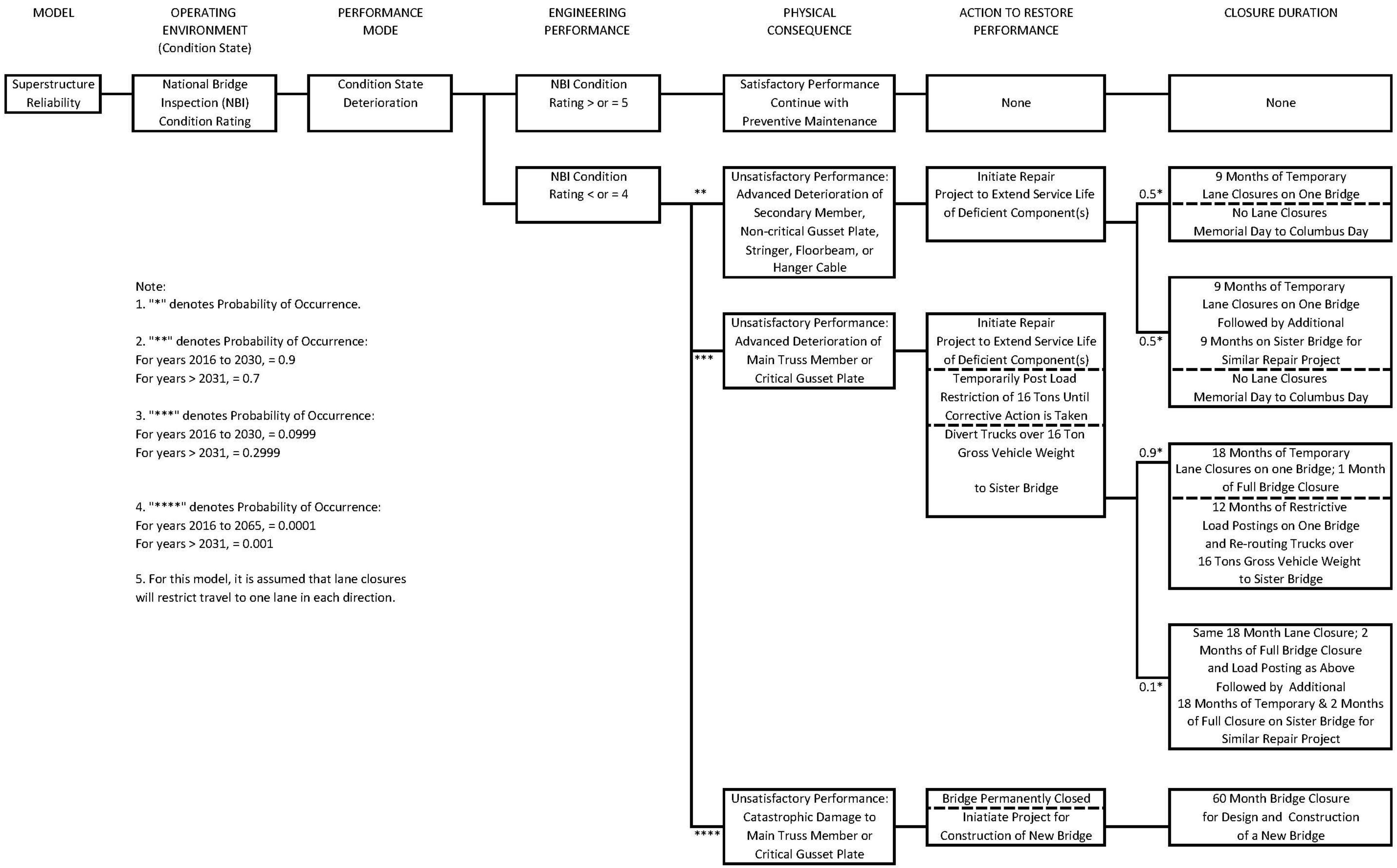
Years 2016 to 2065

Weibull Distribution:
Shape Parameter= 4
Scale Parameter= 156

Where,
h(t) = Failure Rate Function
F(t) = Cumulative Distribution Function
L(t) = Reliability Function
= 1 - F(t)

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2016	1	0.0000	0.0000	1.0000
2017	2	0.0000	0.0000	1.0000
2018	3	0.0000	0.0000	1.0000
2019	4	0.0000	0.0000	1.0000
2020	5	0.0000	0.0000	1.0000
2021	6	0.0000	0.0000	1.0000
2022	7	0.0000	0.0000	1.0000
2023	8	0.0000	0.0000	1.0000
2024	9	0.0000	0.0000	1.0000
2025	10	0.0000	0.0000	1.0000
2026	11	0.0000	0.0000	1.0000
2027	12	0.0000	0.0000	1.0000
2028	13	0.0000	0.0000	1.0000
2029	14	0.0000	0.0001	0.9999
2030	15	0.0000	0.0001	0.9999
2031	16	0.0000	0.0001	0.9999
2032	17	0.0000	0.0001	0.9999
2033	18	0.0000	0.0002	0.9998
2034	19	0.0000	0.0002	0.9998
2035	20	0.0001	0.0003	0.9997
2036	21	0.0001	0.0003	0.9997
2037	22	0.0001	0.0004	0.9996
2038	23	0.0001	0.0005	0.9995
2039	24	0.0001	0.0006	0.9994
2040	25	0.0001	0.0007	0.9993

YEAR	Time	Weibull	Weibull	Weibull
	t	h(t)	F(t)	L(t)
2041	26	0.0001	0.0008	0.9992
2042	27	0.0001	0.0009	0.9991
2043	28	0.0001	0.0010	0.9990
2044	29	0.0002	0.0012	0.9988
2045	30	0.0002	0.0014	0.9986
2046	31	0.0002	0.0016	0.9984
2047	32	0.0002	0.0018	0.9982
2048	33	0.0002	0.0020	0.9980
2049	34	0.0003	0.0023	0.9977
2050	35	0.0003	0.0025	0.9975
2051	36	0.0003	0.0028	0.9972
2052	37	0.0003	0.0032	0.9968
2053	38	0.0004	0.0035	0.9965
2054	39	0.0004	0.0039	0.9961
2055	40	0.0004	0.0043	0.9957
2056	41	0.0005	0.0048	0.9952
2057	42	0.0005	0.0052	0.9948
2058	43	0.0005	0.0058	0.9942
2059	44	0.0006	0.0063	0.9937
2060	45	0.0006	0.0069	0.9931
2061	46	0.0007	0.0075	0.9925
2062	47	0.0007	0.0082	0.9918
2063	48	0.0007	0.0089	0.9911
2064	49	0.0008	0.0097	0.9903
2065	50	0.0008	0.0105	0.9895



Note:
1. "*" denotes Probability of Occurrence.

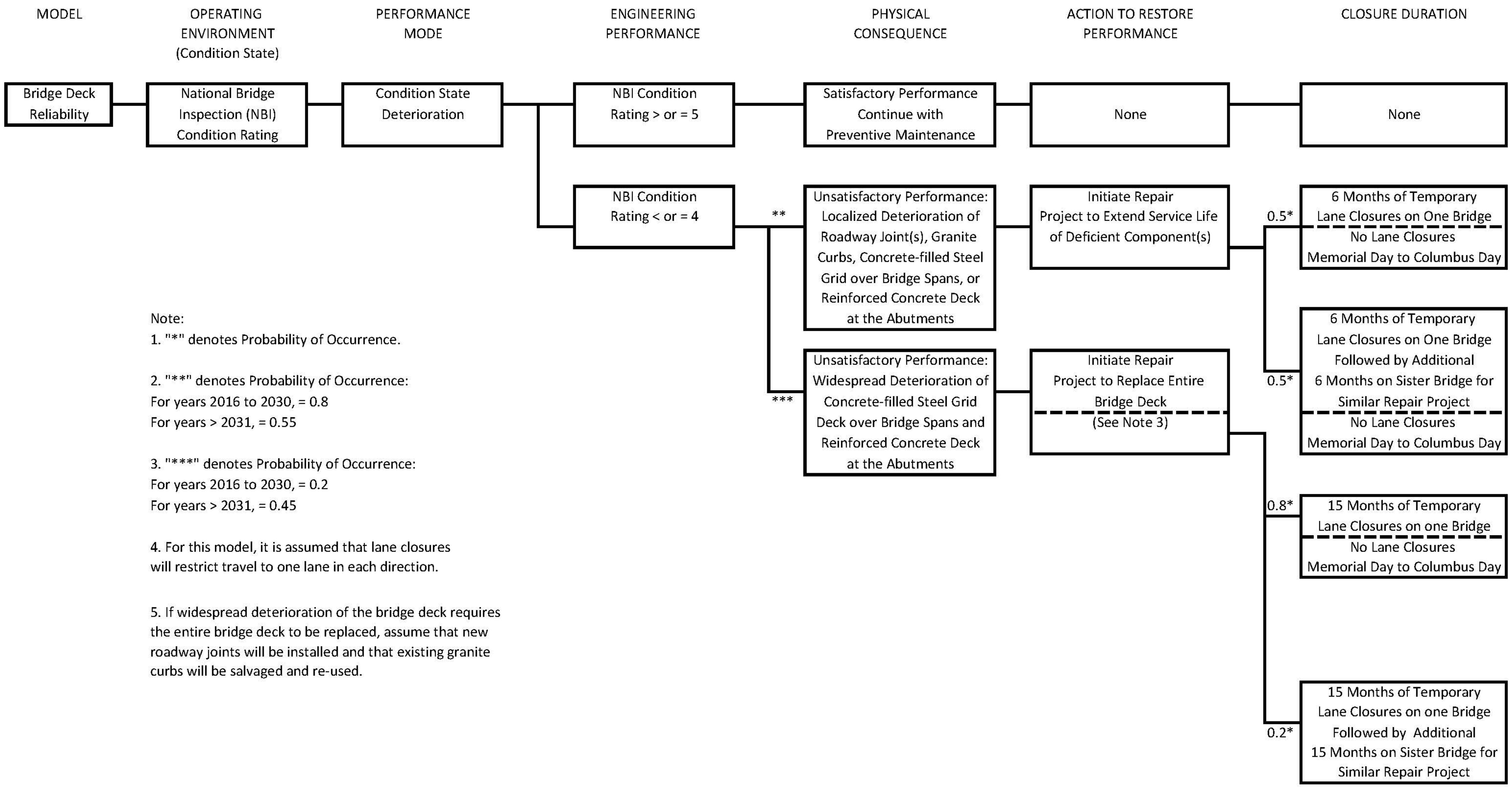
2. "**" denotes Probability of Occurrence:
For years 2016 to 2030, = 0.9
For years > 2031, = 0.7

3. "***" denotes Probability of Occurrence:
For years 2016 to 2030, = 0.0999
For years > 2031, = 0.2999

4. "****" denotes Probability of Occurrence:
For years 2016 to 2065, = 0.0001
For years > 2031, = 0.001

5. For this model, it is assumed that lane closures will restrict travel to one lane in each direction.

FIGURE 8-4: SUPERSTRUCTURE DETERIORATION MODEL-EVENT TREE



Note:
1. "*" denotes Probability of Occurrence.

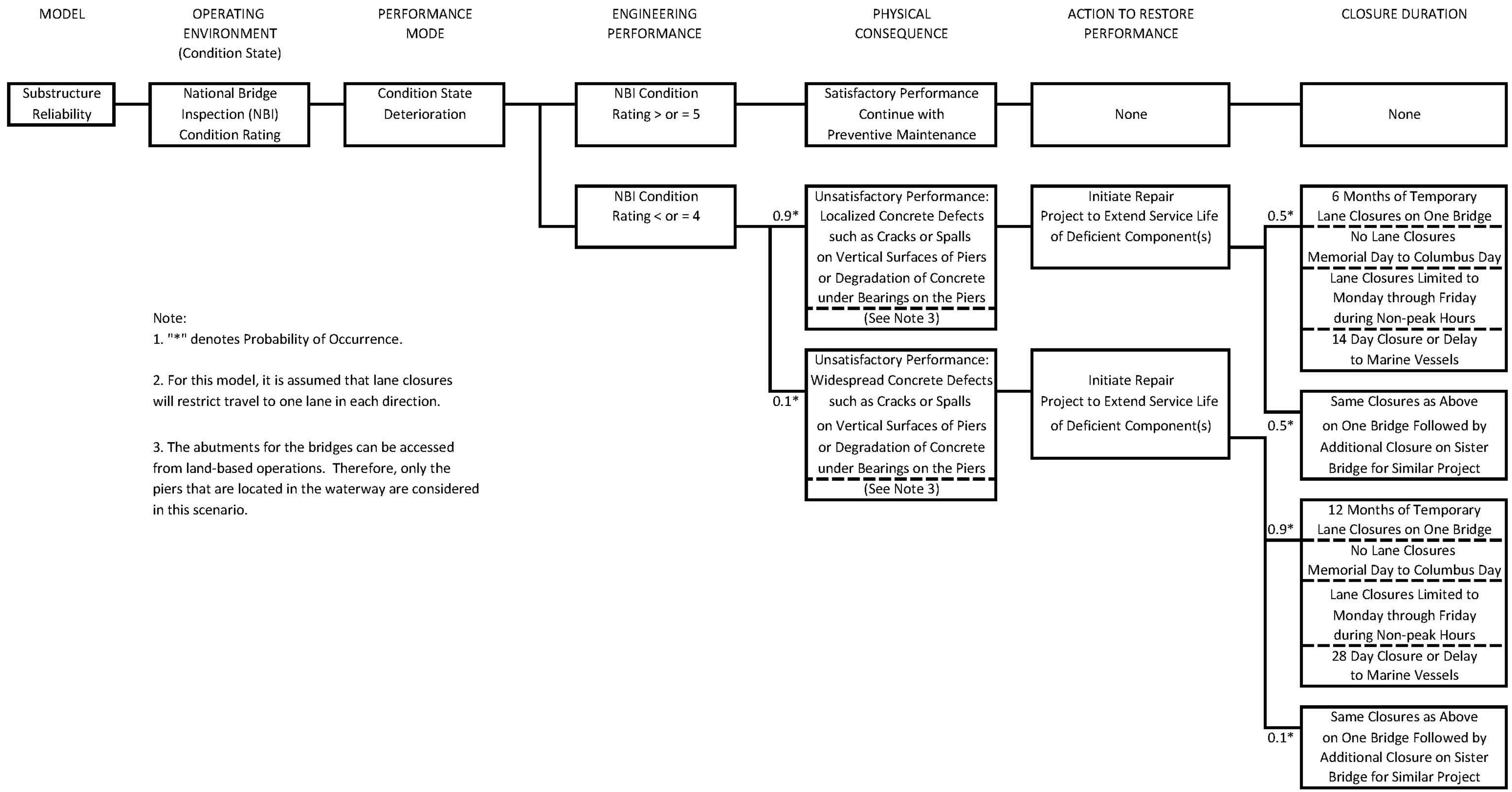
2. "**" denotes Probability of Occurrence:
For years 2016 to 2030, = 0.8
For years > 2031, = 0.55

3. "***" denotes Probability of Occurrence:
For years 2016 to 2030, = 0.2
For years > 2031, = 0.45

4. For this model, it is assumed that lane closures will restrict travel to one lane in each direction.

5. If widespread deterioration of the bridge deck requires the entire bridge deck to be replaced, assume that new roadway joints will be installed and that existing granite curbs will be salvaged and re-used.

FIGURE 8-5: BRIDGE DECK DETERIORATION MODEL-EVENT TREE



Note:
1. "*" denotes Probability of Occurrence.

2. For this model, it is assumed that lane closures will restrict travel to one lane in each direction.

3. The abutments for the bridges can be accessed from land-based operations. Therefore, only the piers that are located in the waterway are considered in this scenario.

FIGURE 8-6: SUBSTRUCTURE DETERIORATION MODEL-EVENT TREE

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f. "Main Truss and Approach Truss Exterior Gusset Plate", Parsons Brinckerhoff, April 2011

g. "2015 Routine Inspection Report of the Sagamore Bridge Over the Cape Cod Canal", Transystems, Volumes I & II, February 3, 2016

h. "2017 Routine Inspection Report of the Sagamore Bridge Over the Cape Cod Canal", Transystems, Volumes I & II, January, 2018

i. "2014 Routine Inspection Report of the Bourne Bridge Over the Cape Cod Canal", Transystems, Volumes I & II,February 17, 2015

j. "2016 Routine Inspection Report of the Bourne Bridge Over the Cape Cod Canal", Transystems, Volumes I & II,February 24, 2017

k. Corrosion Measurements, New England District, March 2013

l. "Vertical-Lift Railroad Bridge Major Rehabilitation Evaluation Report", New England District, May 1997

m. "Atmospheric Corrosion Resistance of Structural Steels", Albrecht and Hall, ASCE Journal of Material in Civil Engineering, February 2003

n. FHWA-HIF-16-002-VOL. 19, "Corrosion Protection of Steel Bridges", December 2015

o. "Capacity Loss Due to Corrosion in Steel-Girder Bridges", Kayser and Nowak, ASCE Journal of Structural Engineering, June 1989

p. ISO. (2012). "Standard 9223 – Corrosion of Metals and Alloys – Corrosivity of Atmospheres – Classification, Determination, and Estimation." International Standards Organization (ISO). Geneva, Switzerland.

ATTACHMENT A

BOURNE & SAGAMORE BRIDGES

DRAWINGS AND PHOTOGRAPHS

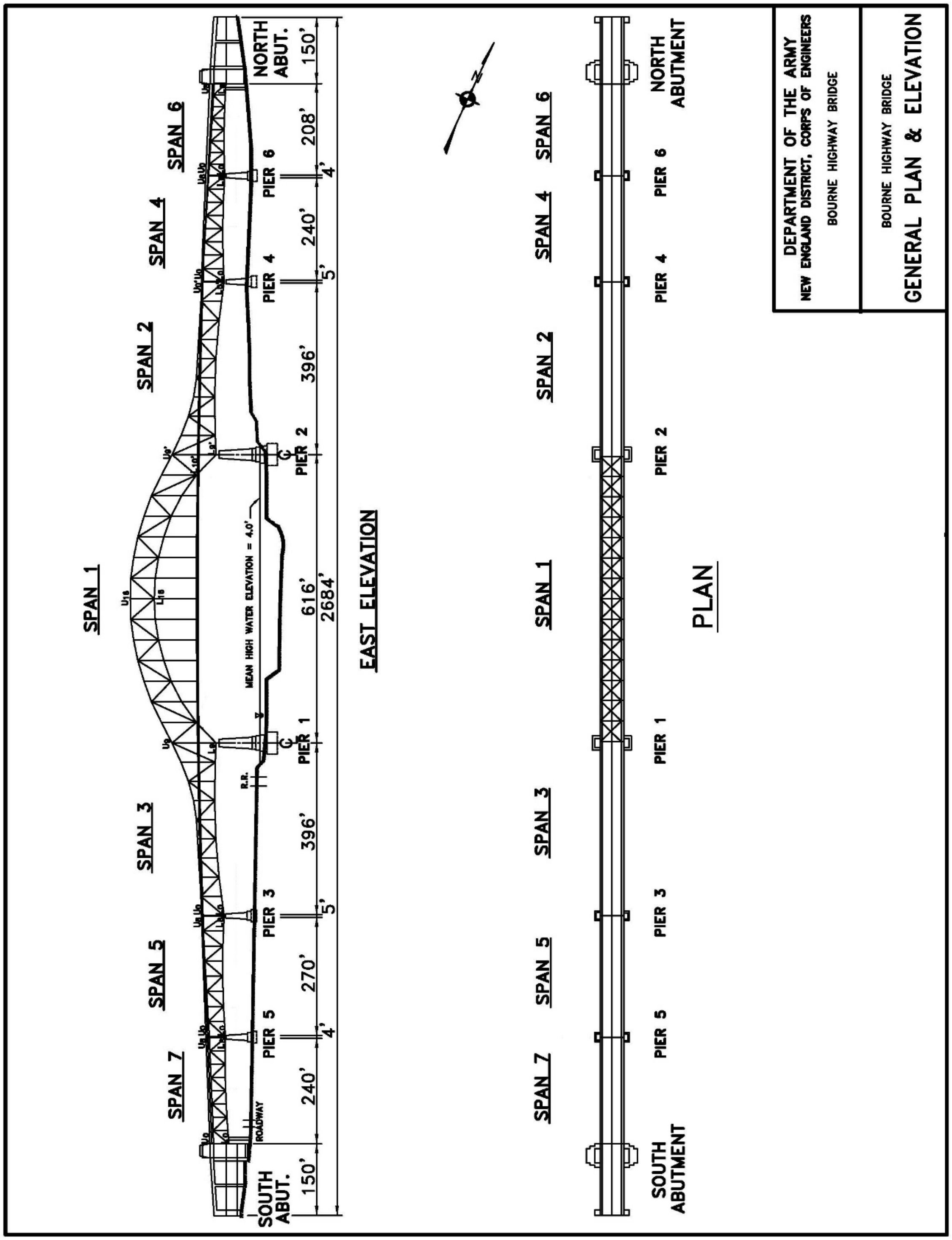
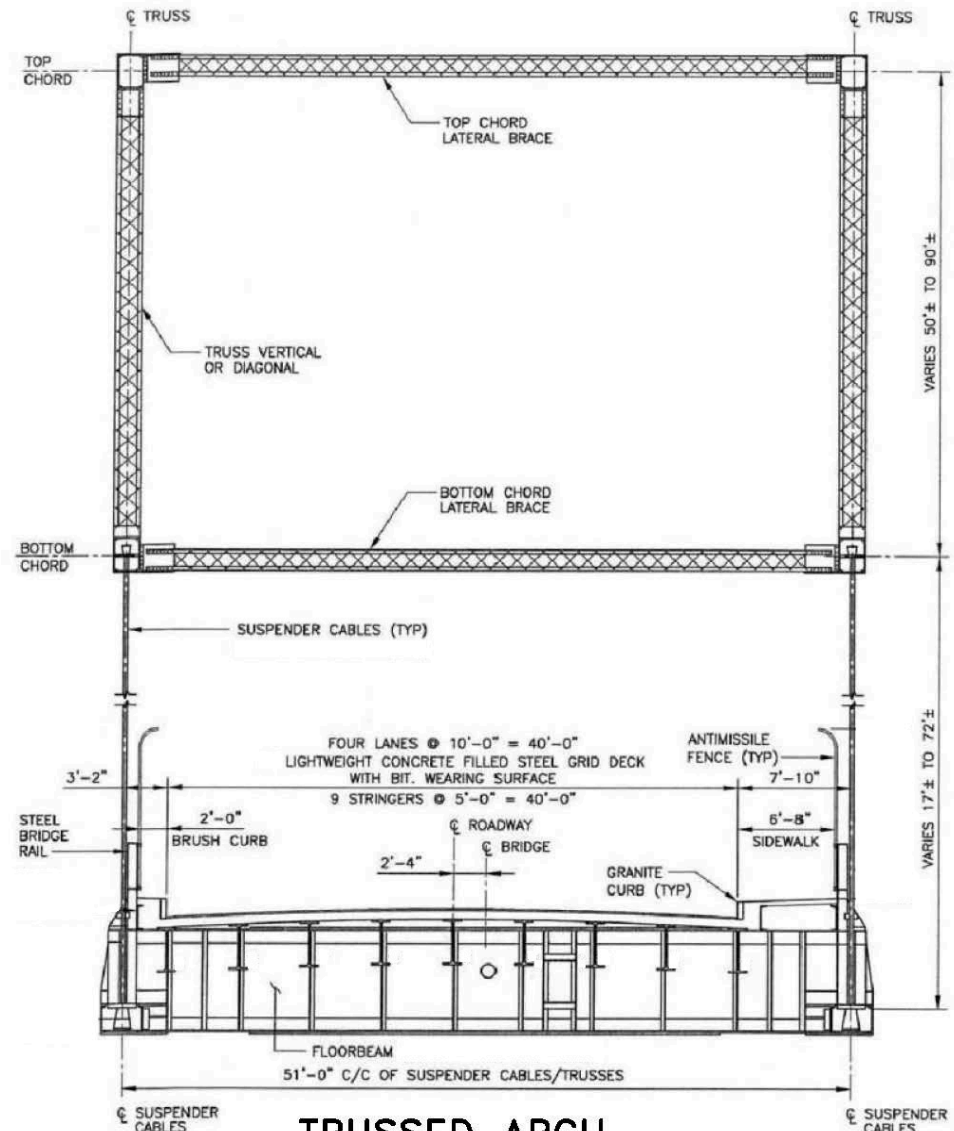

DEPARTMENT OF THE ARMY
NEW ENGLAND DISTRICT, CORPS OF ENGINEERS
BOURNE HIGHWAY BRIDGE
BOURNE HIGHWAY BRIDGE
GENERAL PLAN & ELEVATION
EAST ELEVATION
PLAN
SPAN 1
SPAN 2
SPAN 3
SPAN 4
SPAN 5
SPAN 6
SPAN 7
PIER 1
PIER 2
PIER 3
PIER 4
PIER 5
PIER 6
SOUTH ABUT.
NORTH ABUT.
SOUTH ABUTMENT
NORTH ABUTMENT
MEAN HIGH WATER ELEVATION = 4.0'
ROADWAY
R.R.
150'
240'
4'
270'
5'
396'
616'
2684'
396'
5'
240'
4'
208'
150'

FIG. A - 1



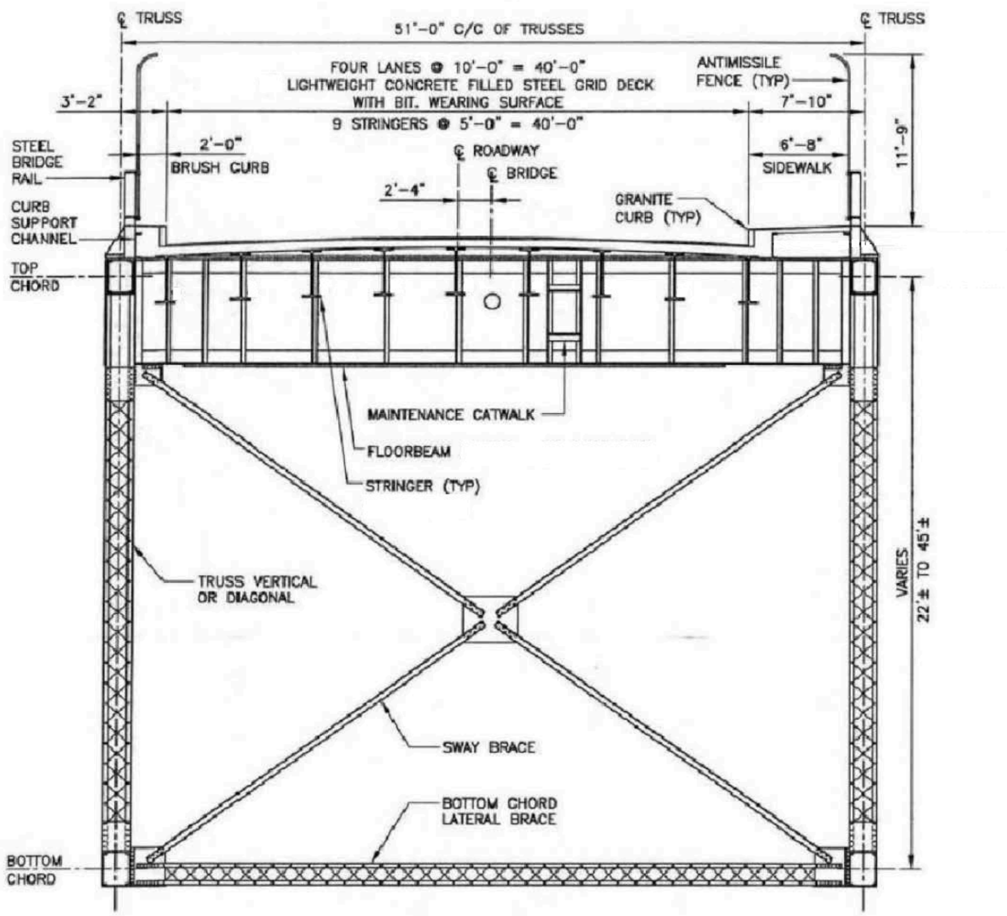
TRUSSED ARCH TYPICAL SECTION (SPAN 1)

(LOOKING SOUTH)

DEPARTMENT OF THE ARMY
NEW ENGLAND DISTRICT, CORPS OF ENGINEERS
BOURNE HIGHWAY BRIDGE

BOURNE HIGHWAY BRIDGE
TRUSSED ARCH TYPICAL SECTION

FIG. A - 2

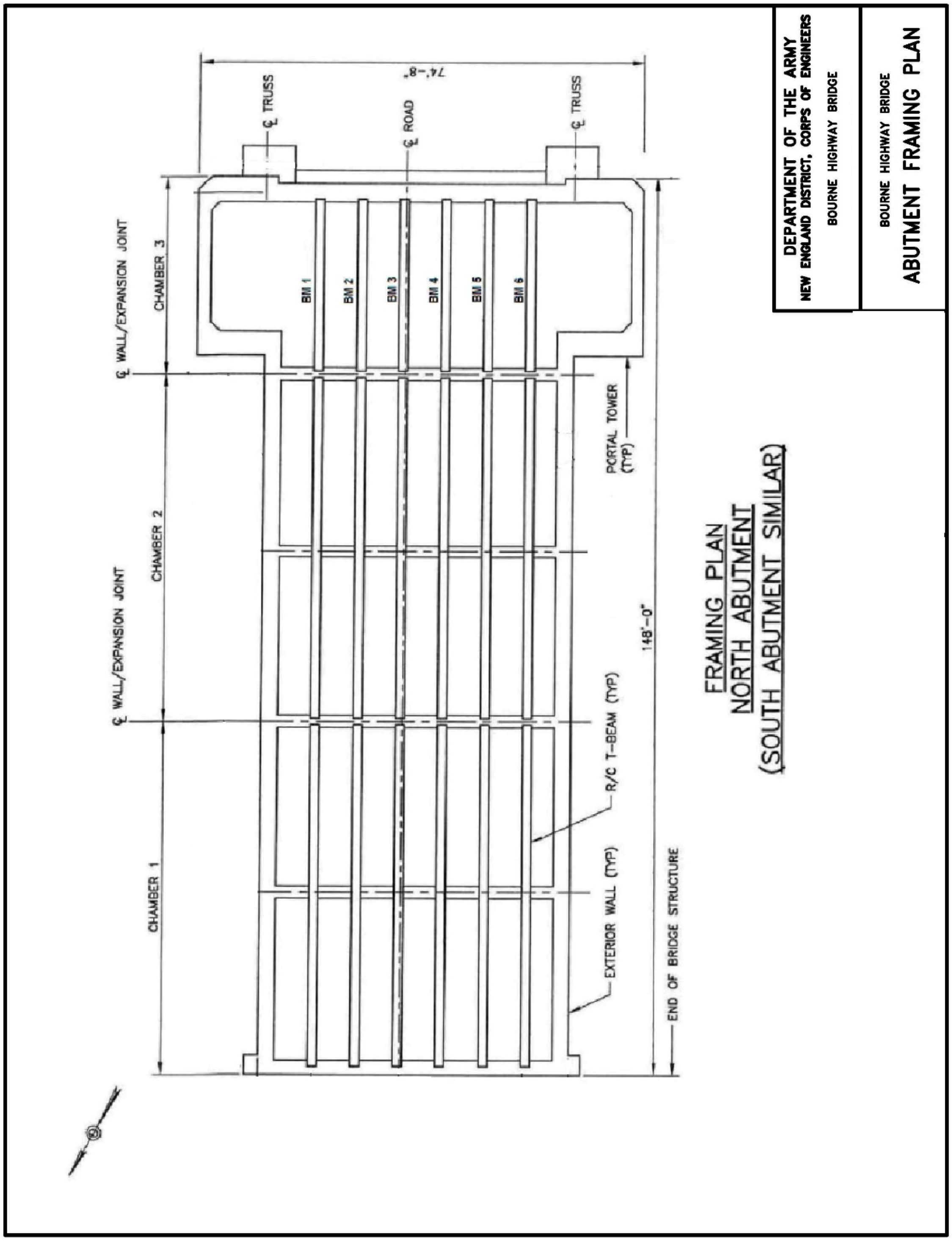


DECK TRUSS
TYPICAL SECTION
(SPANS 2 & 3)
(LOOKING SOUTH)

DEPARTMENT OF THE ARMY
NEW ENGLAND DISTRICT, CORPS OF ENGINEERS
BOURNE HIGHWAY BRIDGE

BOURNE HIGHWAY BRIDGE
DECK TRUSS
TYPICAL SECTION

FIG. A - 3



DEPARTMENT OF THE ARMY
NEW ENGLAND DISTRICT, CORPS OF ENGINEERS
BOURNE HIGHWAY BRIDGE
BOURNE HIGHWAY BRIDGE
ABUTMENT FRAMING PLAN
FRAMING PLAN
NORTH ABUTMENT
(SOUTH ABUTMENT SIMILAR)
74'-8"
℄ TRUSS
℄ ROAD
℄ TRUSS
℄ WALL/EXPANSION JOINT
℄ WALL/EXPANSION JOINT
CHAMBER 3
CHAMBER 2
CHAMBER 1
BM 1
BM 2
BM 3
BM 4
BM 5
BM 6
PORTAL TOWER (TYP)
R/C T-BEAM (TYP)
EXTERIOR WALL (TYP)
END OF BRIDGE STRUCTURE
148'-0"

FIG. A - 4

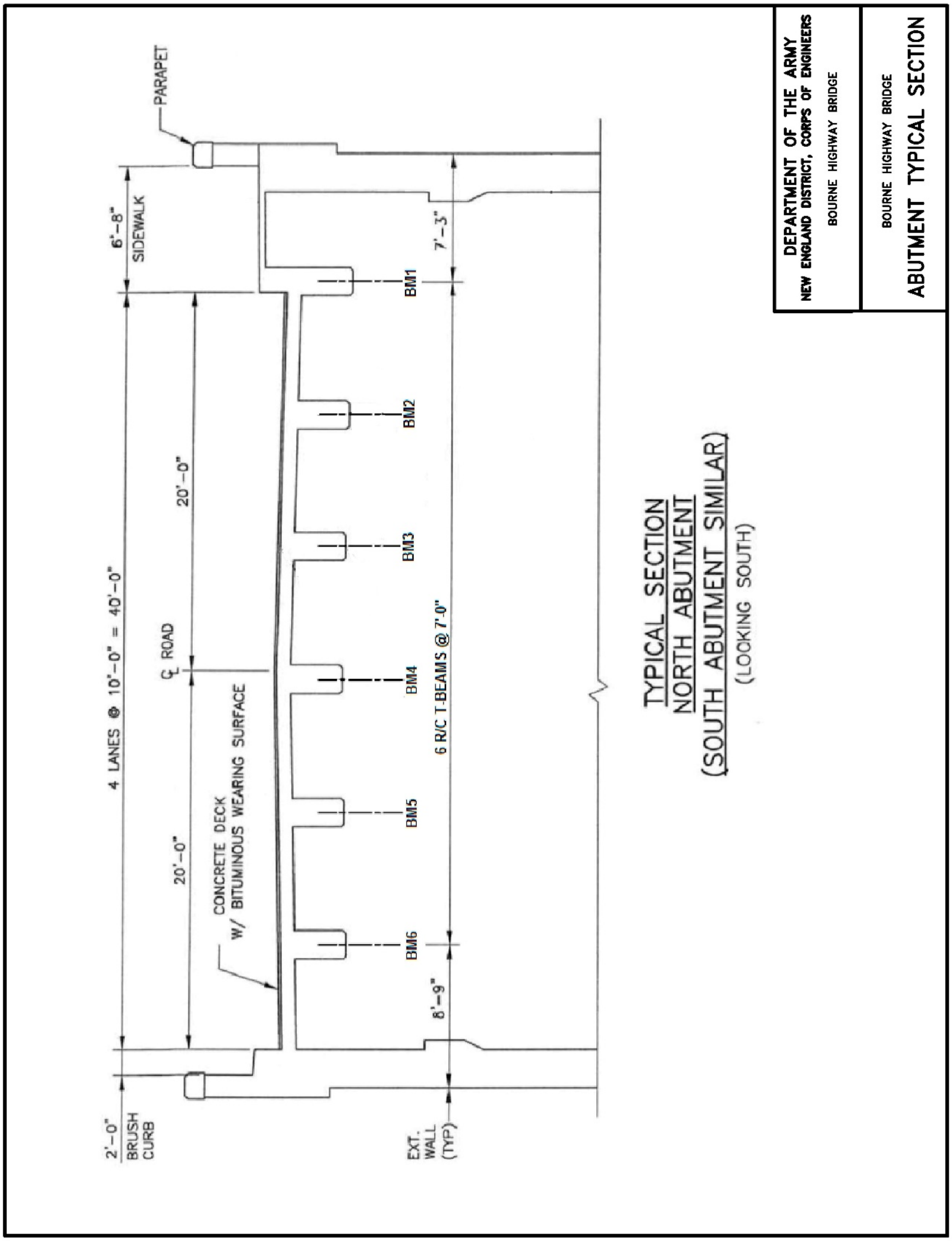

DEPARTMENT OF THE ARMY
NEW ENGLAND DISTRICT, CORPS OF ENGINEERS
BOURNE HIGHWAY BRIDGE
BOURNE HIGHWAY BRIDGE
ABUTMENT TYPICAL SECTION
TYPICAL SECTION
NORTH ABUTMENT
(SOUTH ABUTMENT SIMILAR)
(LOOKING SOUTH)
PARAPET
6'-8"
SIDEWALK
7'-3"
4 LANES @ 10'-0" = 40'-0"
20'-0"
20'-0"
℄ ROAD
CONCRETE DECK
W/ BITUMINOUS WEARING SURFACE
BM1
BM2
BM3
BM4
BM5
BM6
6 R/C T-BEAMS @ 7'-0"
8'-9"
2'-0"
BRUSH
CURB
EXT.
WALL
(TYP)

FIG. A - 5

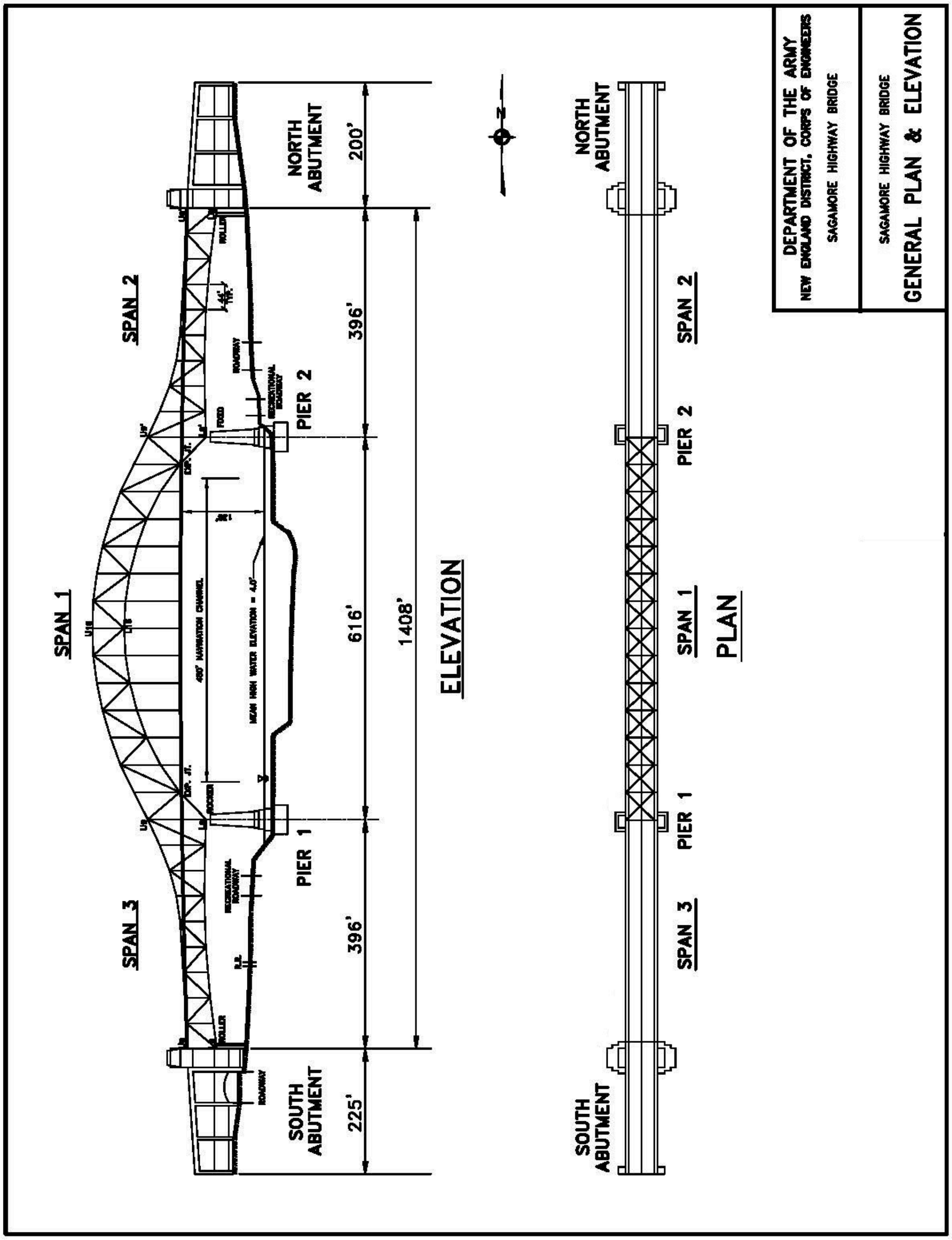
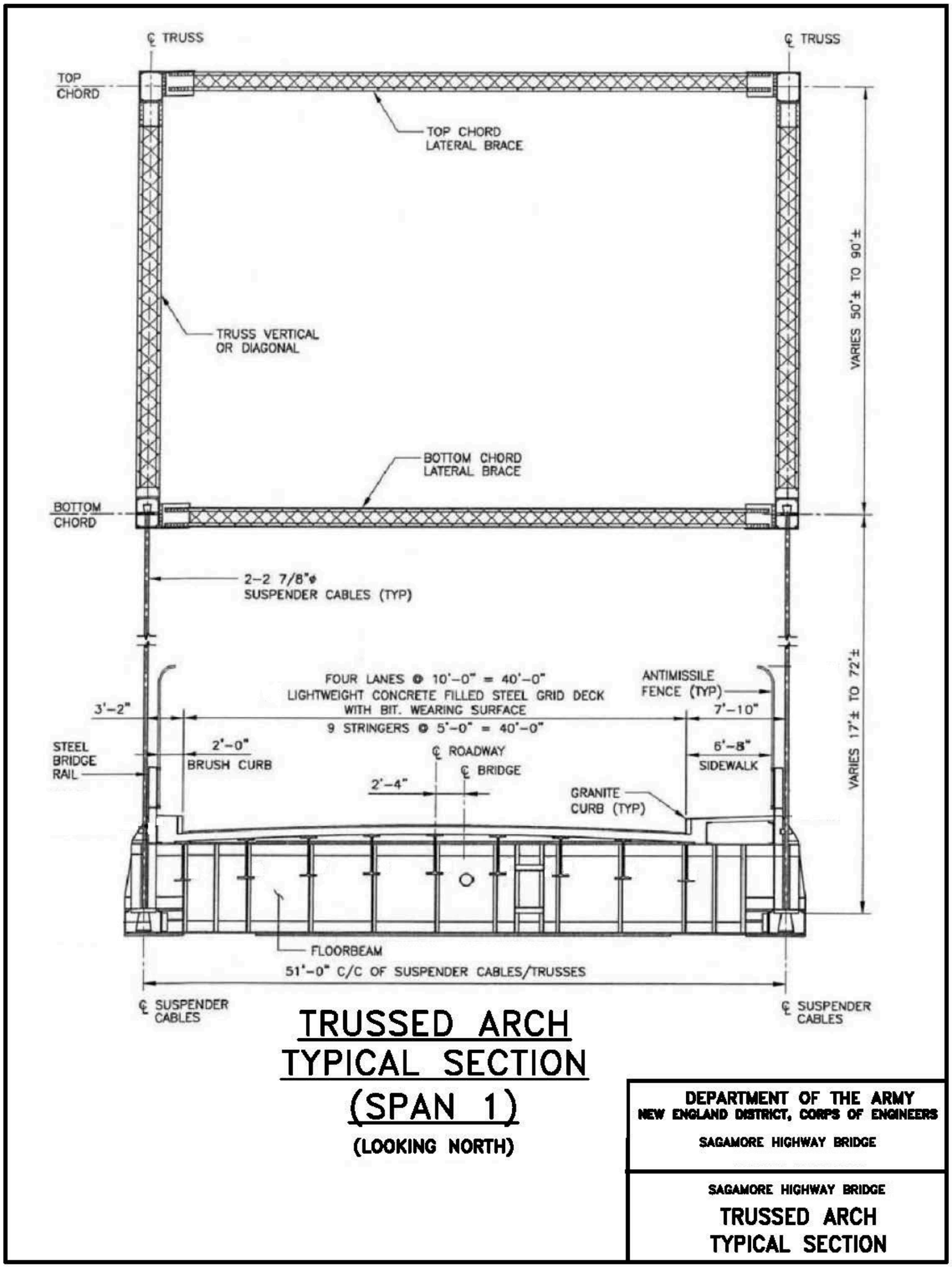
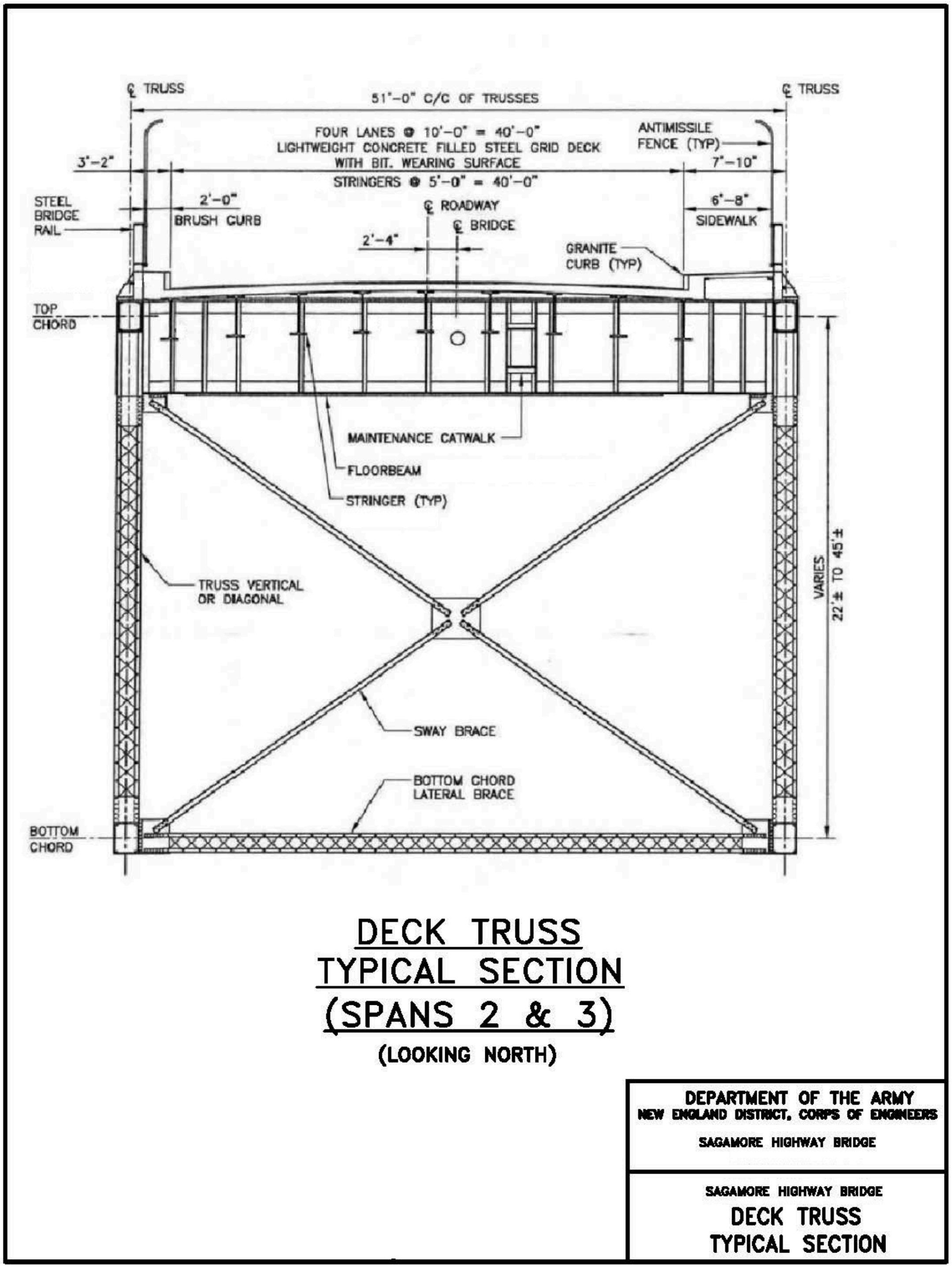

DEPARTMENT OF THE ARMY
NEW ENGLAND DISTRICT, CORPS OF ENGINEERS
SAGAMORE HIGHWAY BRIDGE
SAGAMORE HIGHWAY BRIDGE
GENERAL PLAN & ELEVATION
ELEVATION
PLAN
SPAN 1
SPAN 2
SPAN 3
PIER 1
PIER 2
NORTH ABUTMENT
SOUTH ABUTMENT
200'
396'
616'
396'
225'
1408'
ROLLER
ROCKER
FIXED
EXP. JT.
ROADWAY
RECREATIONAL ROADWAY
MEAN HIGH WATER ELEVATION = 4.5'
N

FIG. A - 6



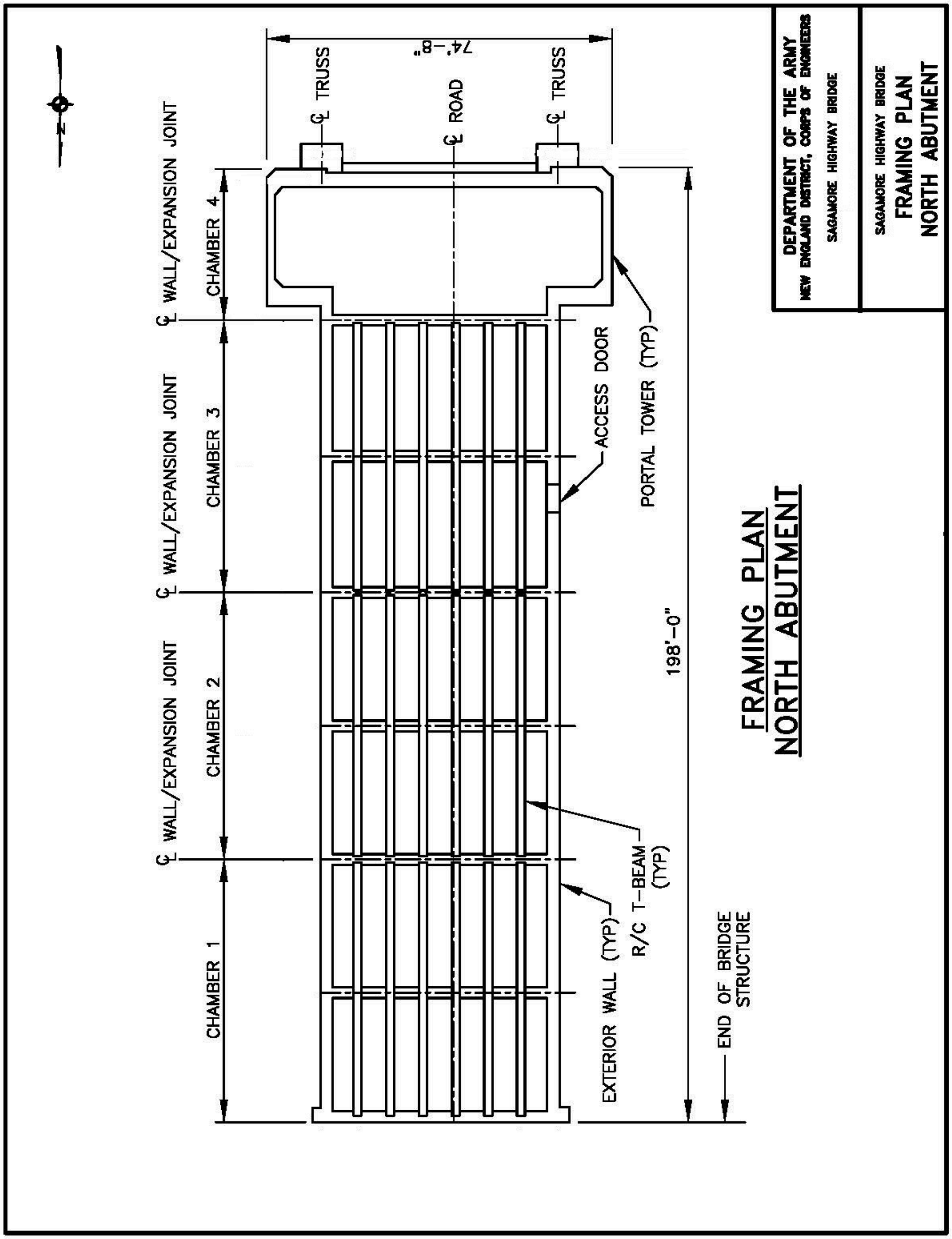
℄ TRUSS
℄ TRUSS
TOP CHORD
TOP CHORD LATERAL BRACE
TRUSS VERTICAL OR DIAGONAL
VARIES 50'± TO 90'±
BOTTOM CHORD LATERAL BRACE
BOTTOM CHORD
2-2 7/8"φ SUSPENDER CABLES (TYP)
FOUR LANES @ 10'-0" = 40'-0"
LIGHTWEIGHT CONCRETE FILLED STEEL GRID DECK WITH BIT. WEARING SURFACE
9 STRINGERS @ 5'-0" = 40'-0"
ANTIMISSILE FENCE (TYP)
3'-2"
7'-10"
STEEL BRIDGE RAIL
2'-0"
BRUSH CURB
℄ ROADWAY
℄ BRIDGE
6'-8"
SIDEWALK
2'-4"
GRANITE CURB (TYP)
VARIES 17'± TO 72'±
FLOORBEAM
51'-0" C/C OF SUSPENDER CABLES/TRUSSES
℄ SUSPENDER CABLES
℄ SUSPENDER CABLES
TRUSSED ARCH TYPICAL SECTION (SPAN 1)
(LOOKING NORTH)
DEPARTMENT OF THE ARMY
NEW ENGLAND DISTRICT, CORPS OF ENGINEERS
SAGAMORE HIGHWAY BRIDGE
SAGAMORE HIGHWAY BRIDGE
TRUSSED ARCH TYPICAL SECTION

FIG. A - 7



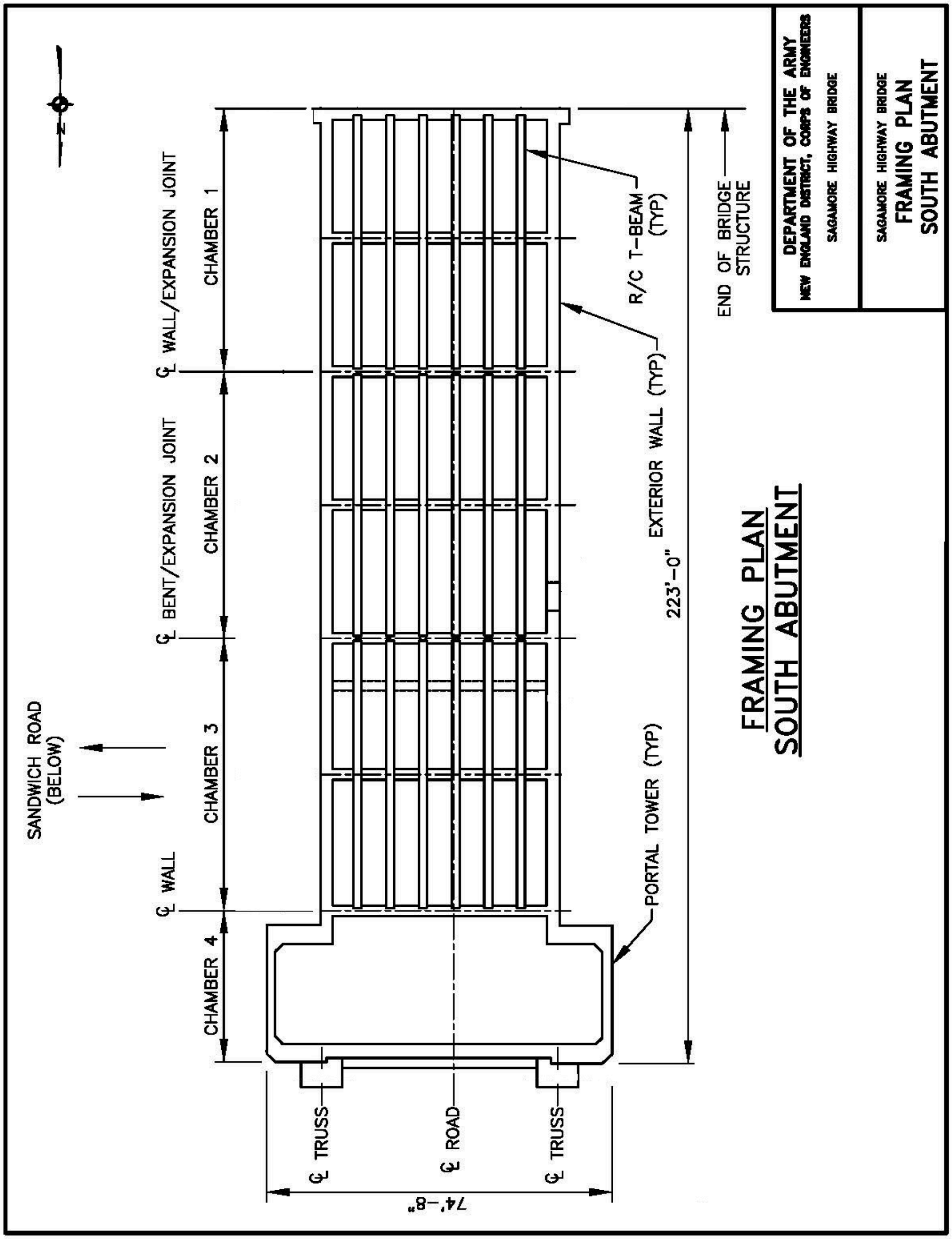
℄ TRUSS
51'-0" C/C OF TRUSSES
℄ TRUSS
FOUR LANES @ 10'-0" = 40'-0"
LIGHTWEIGHT CONCRETE FILLED STEEL GRID DECK
WITH BIT. WEARING SURFACE
STRINGERS @ 5'-0" = 40'-0"
ANTIMISSILE FENCE (TYP)
3'-2"
7'-10"
STEEL BRIDGE RAIL
2'-0"
BRUSH CURB
℄ ROADWAY
℄ BRIDGE
6'-8"
SIDEWALK
2'-4"
GRANITE CURB (TYP)
TOP CHORD
MAINTENANCE CATWALK
FLOORBEAM
STRINGER (TYP)
TRUSS VERTICAL OR DIAGONAL
VARIES 22'± TO 45'±
SWAY BRACE
BOTTOM CHORD LATERAL BRACE
BOTTOM CHORD
DECK TRUSS
TYPICAL SECTION
(SPANS 2 & 3)
(LOOKING NORTH)
DEPARTMENT OF THE ARMY
NEW ENGLAND DISTRICT, CORPS OF ENGINEERS
SAGAMORE HIGHWAY BRIDGE
SAGAMORE HIGHWAY BRIDGE
DECK TRUSS
TYPICAL SECTION

FIG. A - 8



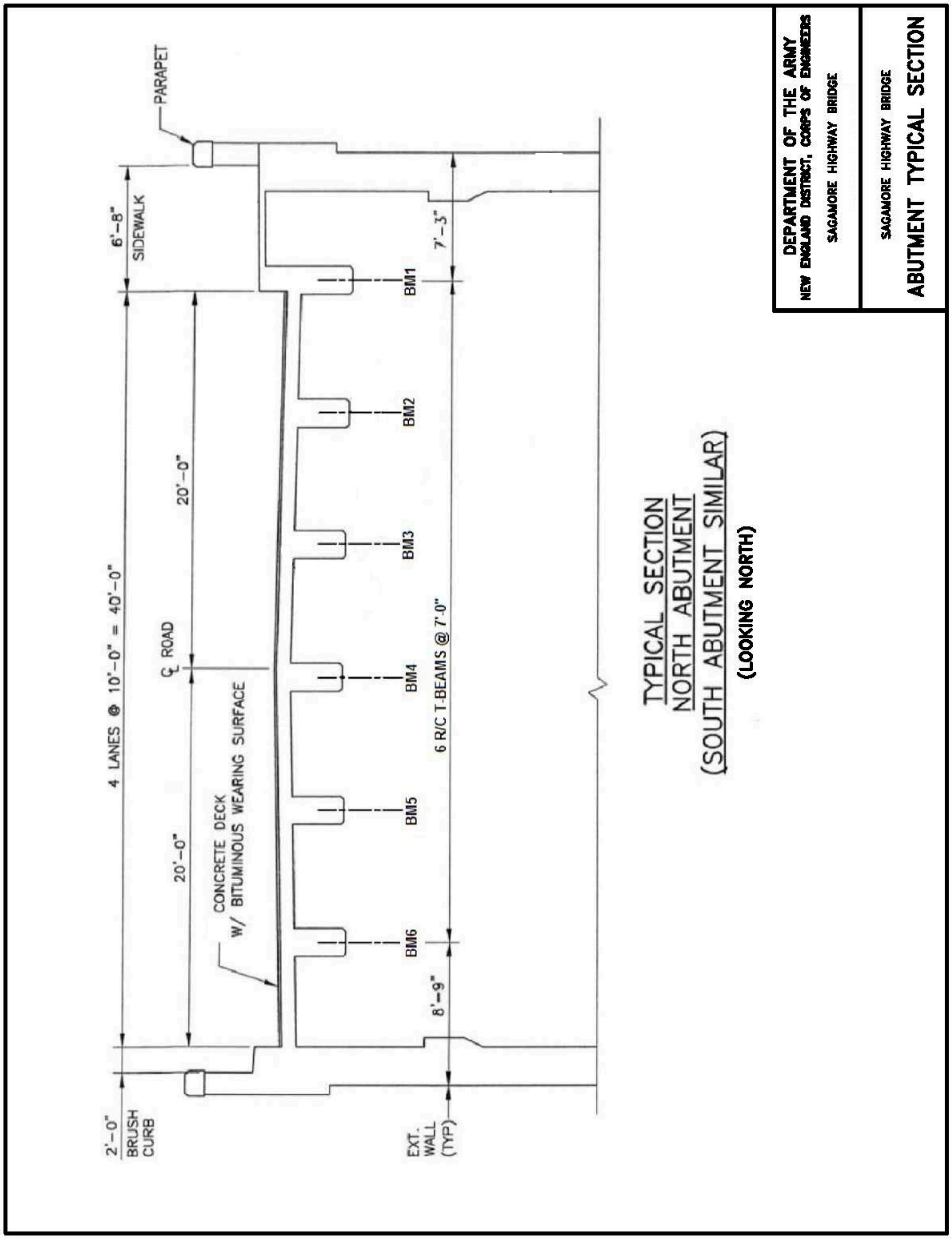
N
74'-8"
℄ TRUSS
℄ ROAD
℄ TRUSS
℄ WALL/EXPANSION JOINT
℄ WALL/EXPANSION JOINT
℄ WALL/EXPANSION JOINT
CHAMBER 4
CHAMBER 3
CHAMBER 2
CHAMBER 1
ACCESS DOOR
PORTAL TOWER (TYP)
198'-0"
R/C T-BEAM (TYP)
EXTERIOR WALL (TYP)
END OF BRIDGE STRUCTURE
FRAMING PLAN
NORTH ABUTMENT
DEPARTMENT OF THE ARMY
NEW ENGLAND DISTRICT, CORPS OF ENGINEERS
SAGAMORE HIGHWAY BRIDGE
SAGAMORE HIGHWAY BRIDGE
FRAMING PLAN
NORTH ABUTMENT

FIG. A - 9



DEPARTMENT OF THE ARMY
NEW ENGLAND DISTRICT, CORPS OF ENGINEERS
SAGAMORE HIGHWAY BRIDGE
SAGAMORE HIGHWAY BRIDGE
FRAMING PLAN
SOUTH ABUTMENT
FRAMING PLAN
SOUTH ABUTMENT
SANDWICH ROAD
(BELOW)
℄ WALL/EXPANSION JOINT
℄ BENT/EXPANSION JOINT
℄ WALL
CHAMBER 1
CHAMBER 2
CHAMBER 3
CHAMBER 4
R/C T-BEAM
(TYP)
EXTERIOR WALL (TYP)
PORTAL TOWER (TYP)
END OF BRIDGE
STRUCTURE
223'-0"
℄ TRUSS
℄ ROAD
℄ TRUSS
74'-8"

FIG. A - 10



PARAPET
6'-8"
SIDEWALK
4 LANES @ 10'-0" = 40'-0"
20'-0"
20'-0"
℄ ROAD
CONCRETE DECK
W/ BITUMINOUS WEARING SURFACE
2'-0"
BRUSH
CURB
BM1
BM2
BM3
BM4
BM5
BM6
7'-3"
6 R/C T-BEAMS @ 7'-0"
8'-9"
EXT.
WALL
(TYP)
TYPICAL SECTION
NORTH ABUTMENT
(SOUTH ABUTMENT SIMILAR)
(LOOKING NORTH)
DEPARTMENT OF THE ARMY
NEW ENGLAND DISTRICT, CORPS OF ENGINEERS
SAGAMORE HIGHWAY BRIDGE
SAGAMORE HIGHWAY BRIDGE
ABUTMENT TYPICAL SECTION

FIG. A - 11

Photo 1 – Bourne Bridge, deteriorated deck of north abutment.

Photo 2 – Bourne Bridge, underside of Pier 3 deck joint.

Photo 3 – Bourne Bridge, dislodged and missing compression seal @ Pier 6 deck joint.

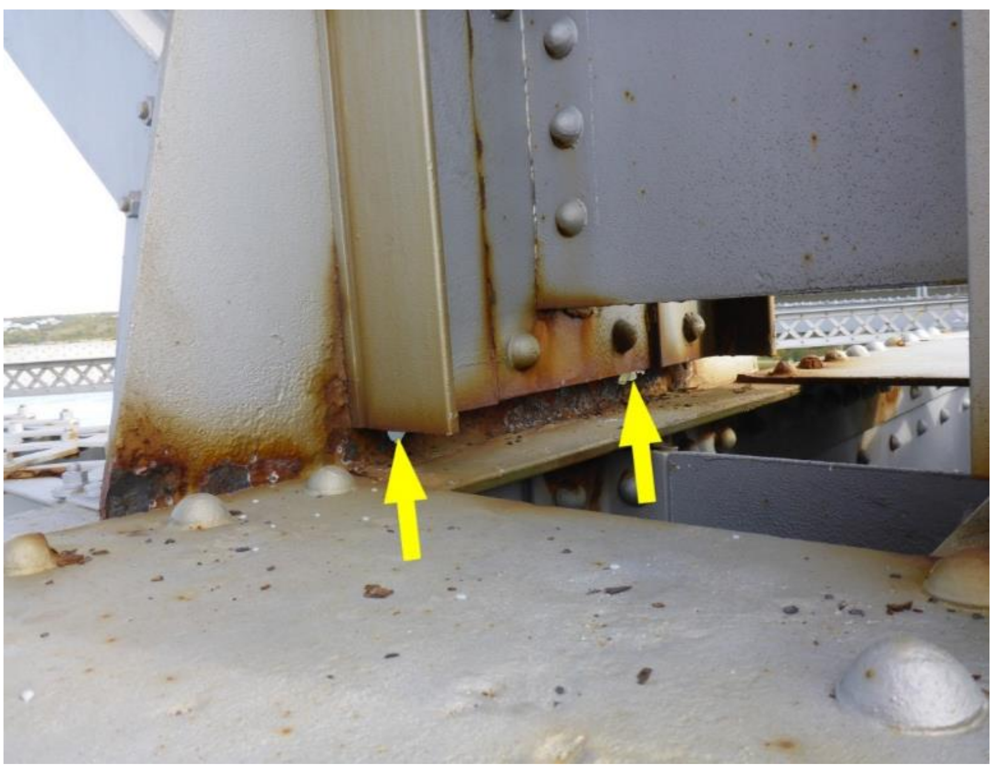


Photo 4 – Bourne Bridge, Span 2, west truss, east gusset plate at L7' with holes along the top of the lower chord member.

Photo 5 – Bourne Bridge, Span 3, west truss, west gusset plate at U0 with thick pack rust along both edges deforming the gusset plate.

Photo 6 – Bourne Bridge, Span 2, east truss, west gusset plate at U6': section loss with active corrosion along the interface with the sidewalk curb channel.

Photo 7 – Bourne Bridge, Span 4, west truss joint L8: Angle welded to the interior face of the east gusset plate (considered a FSD).

Photo 8 – Bourne Bridge, Beam BM1, chamber 2 of the south abutment: The bearing area is undermined resulting in a 33% reduction in bearing area.

Photo 9 – Bourne Bridge, Span 1, north side of floorbeam FB10: Active corrosion with 1/16" loss by full height of the web.

Photo 10 – Bourne Bridge, Span 5, east truss upper chord member U5U6 has up to 1" thick pack rust at the top splice plate with active corrosion on the rivet heads.

Photo 11 – Bourne Bridge, Span 5, west truss diagonal L8U7: Widespread 1/16" deep pitting.

Photo 12 – Bourne Bridge, Span 2, upper lateral bracing at the connection to the east truss at U3': 100% loss by full height of both vertical legs of the bottom flange angles.

Photo 13 – Bourne Bridge, Span 2, east truss, east gusset plate at U0': 1/2" thick pack rust along both edges of the truss vertical member with deforming of the gusset plate.

Photo 14 – Bourne Bridge, Span 3, east bearing at pier 3 has a detached covering and a broken anchor bolt.

Photo 15 – Bourne Bridge, south suspender at east truss joint 14': A 1/16" wide gap by 6" long between two wires, indicative of stage IV corrosion.

Photo 16 – Bourne Bridge, North suspender at west truss joint 13: Area of stage IV corrosion.

Photo 17 – Sagamore Bridge, deteriorated joint header at south abutment.

Repaired in 2018.

Photo 18 – Sagamore Bridge, efflorescence in the underside of the deck in chamber 1 of the south abutment.

Photo 19 – Sagamore Bridge, advanced deterioration to the west gusset plate at truss joint L7 of the east truss in span 3.

Photo 20 – Sagamore Bridge, pack rust along the north edge of truss vertical member deforming the gusset plate at truss joint U0' of the east truss.

Photo 21 – Sagamore Bridge, heavy pitting on the interior face of the east gusset plate at truss joint U0 of the west truss.

Photo 22 – Sagamore Bridge, bent anchor bolt at the south abutment out of plumb 1/4".

Photo 23 – Sagamore Bridge, fatigue sensitive detail utility bracket welded to the north face of the web of floorbeam FB5'.

Photo 24 – Sagamore Bridge, South Abutment on East Face exhibits an area of delaminating concrete 4’x1’ which may pose a future falling hazard.

Photo 25 – Sagamore Bridge, impact damage to the east wind chord.

Photo 26 – Sagamore Bridge, fully cracked weld along the top flange cover plate repair.

Photo 27 – Sagamore Bridge, corrosion hole in the internal longitudinal stiffener plate of east truss upper chord member U0'U1' in span 2.

Photo 28 – Sagamore Bridge, area of pitting to the interior face of east truss vertical member in span 1.

Photo 29 – Sagamore Bridge, interior view of the south connection plate of sway brace with advanced deterioration.

Photo 30 – Sagamore Bridge, pack rust between the upper connection plate and the upper lateral bracing between truss joints at U1 in span 3.

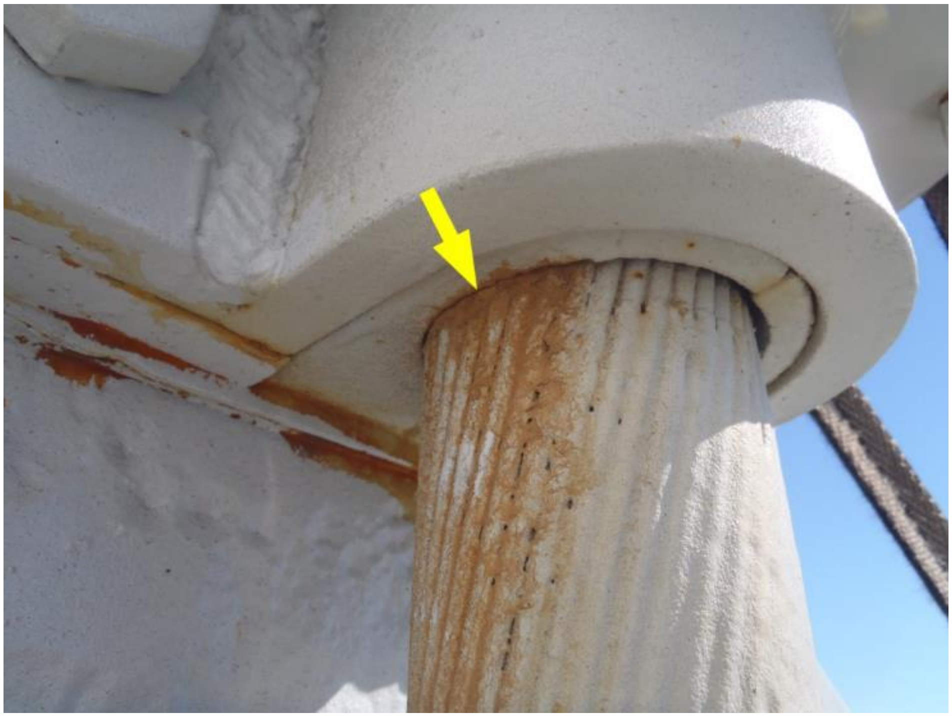


Photo 31 – Sagamore Bridge, misalignment with rubbing between the collar assembly and the south suspender cable at truss joint 15 of the east truss.

Photo 32 – Sagamore Bridge, area of stage III corrosion on the south suspender at east truss joint 12 between the top and bottom rails of the railing.

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CAPE COD CANAL HIGHWAY BRIDGES

BOURNE, MASSACHUSETTS

MAJOR REHABILITATION EVALUATION REPORT

APPENDIX B

PROJECT HISTORY

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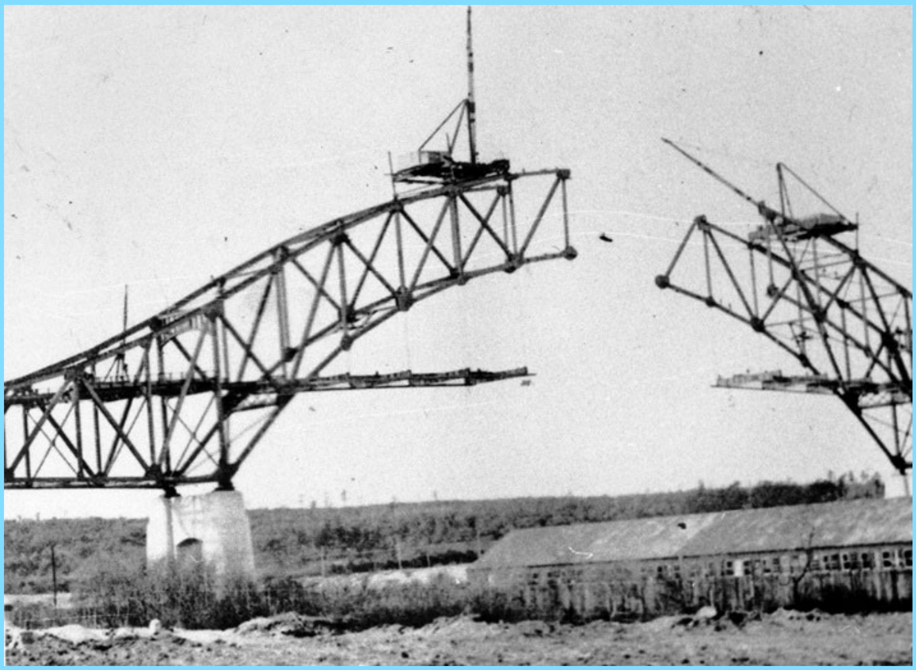

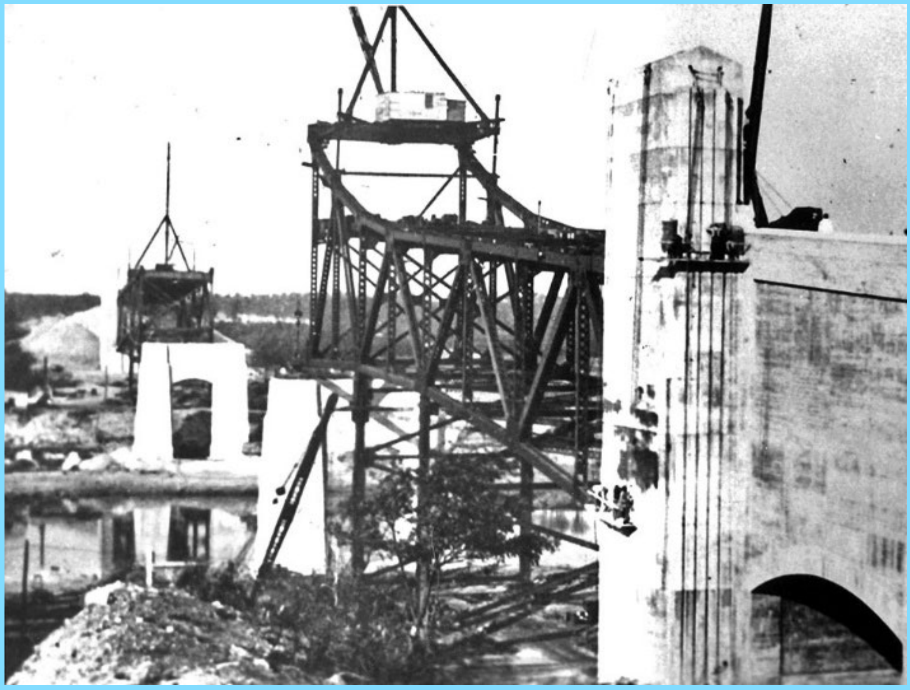

CAPE COD CANAL HIGHWAY BRIDGES
BOURNE, MASSACHUSETTS

MAJOR REHABILITATION EVALUATION REPORT

APPENDIX B – PROJECT HISTORY

Project Study, Authorization and Construction History

The route of the present Cape Cod Canal between the heads of Cape Cod Bay (formerly called Barnstable Bay) and Buzzards Bay was a trade route in colonial times as far back as the 1620s. The Massachusetts Bay Colony and later the Commonwealth of Massachusetts repeatedly studied the idea of a canal in the 1690s, 1770s, 1790s, and through much of the 19th Century. The earliest reports of surveys for a canal by the Corps of Engineers are contained in the following documents.

Report Called for by:	Report – U.S. Serial Set and Date
Act of 30 April 1824	Senate Document #32, 18th Congress, 2d Session, 14 February 1825
	House Document #174, 19th Congress, 1st Session, 2 March 1826
House Resolution of 2 January 1827	House Document #54, 21st Congress, 1st Session, 8 February 1830
Senate Commerce Committee Letter of 25 April 1870	Senate Miscellaneous Document #145, 41st Congress, 2d Session, 26 May 1870
River & Harbor Act of 3 March 1881	Senate Executive Document #104, 47th Congress, 1st Session, 14 February 1882
River & Harbor Act of 3 June 1896	House Document #311, 54th Congress, 2d Session, 24 February 1897
River & Harbor Act of 4 March 1913	House Document #1341, 63rd Congress, 3d Session, 11 December 1914

Different canal routes were considered, including a route from the Atlantic to Barnstable Bay via Nauset Harbor and Rock Harbor, routes from Nantucket Sound to Barnstable Bay via the Bass River or from Hyannis Harbor to Barnstable Harbor, and the more often examined route from Buzzards Bay to Barnstable Bay via the Monument and Scusset Rivers. While all of the afore-mentioned harbors were dredged by either the Commonwealth or the Corps for navigation by their local fleets, only the Buzzards Bay route was ever dredged for establishment of a canal.

On 26 June 1883 the Massachusetts legislature (Chapter 259 Laws of 1883) granted a charter to the Cape Cod Ship Canal Company for construction of a canal following the Monument and Scusset Rivers. Dredging was begun at the eastern end of the cut but was abandoned shortly afterward when funds were exhausted. The corporate charter expired without the Company completing a canal.

On 1 June 1899 the Massachusetts legislature (Chapter 448 Laws of 1899) granted another charter for construction of a canal, this to the Boston, Cape Cod and New York Canal Corporation. Activities under this charter were subject to approval by the Commonwealth through a Joint Board of Railroad and Harbor Commissioners of Massachusetts (and later the Waterways and Public Lands Commission), which reviewed and approved all plans, contracts and finances of the Canal Company. Concerning bridge crossings, Section 14 of MA Chapter 448 states that "said canal company shall provide and maintain in the towns of Bourne and Sandwich, at such points as may be designated by the county commissioners, suitable ferries or bridges across the canal, or a suitable tunnel or tunnels under the same, for passengers and vehicles, to be operated free from tolls, under reasonable rules to be established by the county commissioners, except that the canal company shall not be required to maintain a ferry if a highway bridge or tunnel shall be built at or near any of said points…." Section 15 of MA Chapter 448 states that "said company shall also construct such highways over its location to connect with the bridge or bridges, tunnel or tunnels, and ferries herein provided for, and such other highways as may be necessary to replace the highways destroyed by the construction of said canal …"

Construction of the canal began 22 June 1909 and the Canal was opened to navigation on 4 July 1914 to vessels drawing up to 12 feet. Allowable draft was increased to 20 feet in October 1915, and to 25 feet in April 1916 upon its completion. Although it was not until 25 January 1918 that the State declared the Canal completed in accordance with the company's charter. Tidal assistance was required for passage of these vessels as the canal design depth was 25 feet. The canal channel had a width of 200 to 300 feet in its seaward approaches and 100 to 150 feet through the 7.7 mile long land cut.

Three draw bridges crossed the canal, each with a horizontal clearance of 140 feet between the fenders. A highway bridge was located near the eastern end at Sagamore. A combined highway and trolley bridge was located near the western end at Buzzards Bay, and a railroad bridge was located seaward of the western highway bridge. A ferry crossing was located at Bournedale about midway between the highway bridges.

The Canal Company charter was modified three times by the legislature, as shown below.

Massachusetts Act	Purpose
Chapter 448, 16 April 1899	Act to Incorporate the Boston, Cape Cod and New York Canal Company, with Capital Stock of $6 Million, to Construct and Operate a Canal from Buzzards Bay to Barnstable (Cape Cod) Bay through the Town of Bourne or Sandwich or Both, including such Lands, Highways, Bridges, Tunnels, Breakwaters, Wharves and Vessels as Needed or Required. Company shall Pay Damages for taking and Relocating Railroads and Railroad Crossing. Creates a Joint Board consisting of the Land and Harbor Commission and the Railroad Commission to Review and Approve all Plans, Contracts and Bridge Crossings with Suitable Draw Spans. The Joint Board would Determine of Highway Bridges, Railroad Bridges and Ferries, and Highways and Railroad Lines within the Canal Company Lands.

	Section 14 – Canal Company to Provide and Maintain Highway Bridges, Ferries or Tunnels (and Highways to Connect with such – Section 15) in the Towns of Bourne and Sandwich at Points Determined by the County Commissioners and Free from Tolls. A Ferry will not be Required at any Point where a Bridge or Tunnel has been Provided.
Chapter 476, 17 July 1900	Provides that the Commonwealth may Purchase the Canal by paying the Company the Cost of its Investment and Bonds plus Ten Percent.
Chapter 519, 13 May 1910	Provides that the Joint Commission Created by Chapter 448 (1899) as Amended may Change the Points as it Previously Determined for the Railroad and Highway Crossings of the Cape Cod Canal, and Provided that the Canal Company Five Years from the Date of Enactment to Complete Construction of the Canal.
Chapter 184, 16 April 1917	Creates a Joint Commission consisting of the Commissioners of Public Service and Waterway & Public Lands, the Commissioners of Barnstable County, and the Selectmen of the Towns of Bourne and Sandwich. The Commission may order the Discontinuation of the Bournedale Ferry Service, and may Amend, Modify, or Revoke any Order made under Chapter 448 (1899) as Amended for Construction and Maintenance of a Bridge, Ferry or Tunnel at Bournedale. Provided that a Street Railway Service be First Constructed and Operated along the North side of the Canal between Sagamore, Bournedale and Bourne Villages.

With private construction of the canal underway, Congress again took an interest in the matter and began calling for reports on the subject. The River and Harbor Act of 4 March 1913 called for a report on improving the western approach to the canal, including the removal of Cleveland Ledge. The responding Preliminary Examination, 11 December 1913 and Survey Report, 30 September 1914, both printed in House Document #1341, 63rd Congress, 3d Session, 11 December 1914 were unfavorable to such work as these obstructions were considered easily avoidable by ships transiting the canal.

The River & Harbor Act of 4 March 1915 called for a study of providing a harbor at Onset Bay connected to the western end of the Canal. The responding Preliminary Examination, 5 November 1915, as printed in House Document #810, 64th Congress, 1st Session, 1 March 1916, was unfavorable to adopting such a project, but that issue would be revisited once the canal was acquired by the Federal Government.

Senate Document #279, 65th Congress, 2d Session, 24 September 1918, prepared near the end of World War I (called for by Senate Resolution of 5 July 1918), contains reports on three east coast canals including the Cape Cod Canal. The report by the Department of Commerce, Bureau of Foreign and Domestic Commerce, provides a discussion of the history, development, operation, national defense needs, and features of the three canals and provides estimates for their improvement and takeover by the United States. The report states that on 22 July 1918 possession of the Cape Cod Canal was placed under the control of the Director General of

Railroads by a Presidential Proclamation 1419, 26 December 1917 (40 Stat. 1808), and that operation of the canal was entrusted to the United State Railroad Administration (USRA).

These actions were taken under authority in Section 1 of the Army Appropriations Act of 29 August 1916 (39 Stat. 645) which gave the President power, in time of war, "to take possession and assume control of any system or systems of transportation, or any part thereof, and to utilize the same, to the exclusion as far as may be necessary of all other traffic thereon, for the transfer or transportation of troops, war material and equipment, or for such other purposes connected with the emergency …". Under this authority the USRA took control and operated railroads, coastwise steamship lines, inland waterways, and telephone and telegraph companies seized in the interest of national defense, and entered into compensatory agreements with seized carriers and utilities pursuant to the Federal Control Act of 21 March 1918 (40 Stat. 451). The USRA began operating the Canal on 25 July 1918 and proceeded with maintenance dredging of the Canal to return its controlling depth to the 25-foot design depth. The railroads and other seized properties and concerns were returned to private control on March 1, 1920, under terms of the Transportation Act of 28 February 1920 (41 Stat. 470), and the USRA commenced with liquidation and final settlement of accounts with the owners. Congress however was concurrently examining Federal acquisition of both the Cape Cod and the Chesapeake and Delaware canals, and called for additional studies.

House Document #1768, 65th Congress, 3d Session, 6 February 1919, contains reports on the cost and advisability of purchase and enlargement of the Cape Cod Canal by the Federal Government, as called for by the River and Harbor Act of 8 August 1917 (40 Stat. 250, P.L. 65-37). That Act called on the Secretaries of the Navy, War and Commerce to examine the Canal, appraise its value, make a recommendation for its purchase, and begin negotiations with the owners for its purchase. The reports also included a recommendation to deepen the canal to 30 feet and widen the land cut channel to 200 feet. The Canal Company declined the Government's initial offer of $8,250,000 for the Canal and made a counter-proposal for $13 million. In House Document #1812, 65th Congress, 3d Session, 17 February 1919, (and Senate Report #761, 65th Congress, 3d Session, 25 February 1919), letters from the Railroad Administration and Secretary of War to Congress were printed. The Railroad Administration stated that with the end of the U-Boat threat to shipping that its operation of the Canal was no longer justified. The Secretary of War requested authorization to take possession of the Canal once condemnation proceedings were completed. Proposed language authorizing the purchase not to exceed $10 million was printed in House Document #68, 66th Congress, 1st Session, 2 June 1919.

The Government began condemnation proceedings in U.S. District Court for Massachusetts, and a jury verdict (18 November 1919) set a price of $16.8 million minus $150,000 for maintenance performed by the Railroad Administration. On 1 March 1920 the Federal Government relinquished control of the Canal and attempted to return it to the Canal Company. The Company initially refused to accept return, but agreed to operate the Canal while negotiations continued and to turn-over a portion of excess revenues to the Government (Senate Report #924, 68th Congress, 2d Session, 22 January 1925). On appeal the U.S. 1st Circuit Court set aside the November 1919 judgment (16 February 1921) for error and ordered a new trial.

The Secretary of War and the Canal Company then agreed on a price of $11.5 million on 21 July 1921 and executed a contract on 29 July 1921. House Document #139, 67th Congress, 2d

Session, 12 December 1921 prints letters from the Secretary of War and the Bureau of the Budget, and the proposal from the Canal Company. The purchase was to be $5.5 million cash, plus $6 million for the Government's payment on the value and interest on the Company's bonds. The Canal Company continued operation of the Canal pending Congressional ratification of the purchase contract.

Between 1921 and 1927 Congress repeatedly took up the issue of purchasing the Canal. Numerous hearings were held and bills and committee reports drafted. Concern was expressed with the post-war fiscal limitations, Government interference in commerce, and Federal assumption of business debts. A selection of committee reports outlining the differing House and Senate views on these issues includes:

House Report #1016, 67th Congress, 18 May 1922
House Report #181, 68th Congress, 11 February 1924
Senate Report #924, 68th Congress, 22 January 1925

The River and Harbor Act of 21 January 1927, Section 2 (44 Stat. 1010, P.L. 69-560, H.R. 11616) ratified the contract for purchase of the Canal with certain stipulations limiting the start date for the period for which the Government was responsible for payment of interest on the Company's bonds, plus a requirement for a joint general release of claims by the Government and the Company. The purchase price remained $5.5 million cash, plus $6 million for principal and interest on the bonds, as specified in House Document #719, 69th Congress, 2d Session, 15 February 1927. The first Deficiency Appropriations Act for Fiscal Year 1928, 22 December 1927 (45 Stat.2, P.L.70-2) appropriated the $5.5 million for the cash portion of the purchase. The Second Deficiency Appropriations Act for Fiscal Year 1928, 29 May 1928 (45 Stat.883, P.L.70-563) appropriated the $6 million for the assumption of the Company's bond debts including interest, as specified in House Document #221, 70th Congress, 1st Session, 10 April 1928. Title to the Canal was to pass to the Government on 1 January 1929, although the Government had assumed control and operation of the Canal on 31 March 1928. At that time tolls ceased and the Corps began operation of the canal, bridges and ferry, with maintenance dredging beginning that July (Annual Report of the Chief of Engineers, 1929).

The River and Harbor Act of 3 July 1930 (46 Stat. 918, P.L. 71-520) directed a study be made of the Cape Cod Canal. The reports of the preliminary examination and survey report are printed in House Document #795, 71st Congress, 3d Session, 3 March 1931, and made the following recommendations:

Project Features	District Engineer Recommendations	Division Engineer Recommendations	BERH Recommendations
Channel Depth	35 feet	32 Feet	30 Feet
Locks	2 Locks 110 x 1000 feet, 40 feet over sills	2 Locks 110 x 1000 feet, 40 feet over sills	1 Lock 110 x 1000 feet, 40 feet over sill
Land Cut Width	300 feet	300 feet	250 feet
Sea Cut Width	500 feet	500 feet	400 feet
Channel Width Seaward of Wings Neck with a Straighter Alignment	700 feet	700 feet	700 feet
Highway Bridges	One fixed high-level	One fixed high-level	One fixed high-level
Railroad Bridge	One new drawspan	One new drawspan	One new drawspan
Small craft harbors	Harbor of Refuge at East end and 15-foot harbor at Onset Bay	Harbor of Refuge at East end and 15-foot harbor at Onset Bay	15-foot harbor at Onset Bay

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The Chief of Engineers concurred in the recommendations of the Board of Engineers for Rivers and Harbors. These reports cite a peak summer bridge traffic volume of "more than 1000 cars per hour over each bridge." Plans for a new highway crossing considered a central location for either a new single six-lane high-level fixed highway bridge or a single tunnel. It was also considered that the proposed single high-level highway bridge might also include a railroad deck, but absent that a new railroad bridge with movable span and greater horizontal channel clearance would be needed. A statement of traffic volumes for the two highway bridges from the 1930 survey report is as follows:

Winter average daily number of cars	1,200
Winter average monthly number of cars	36,000
Summer average daily number of cars	4,700
Summer average monthly number of cars	142,000
Summer peak Sunday number of cars	9,400

Concerning the Bournedale Ferry, the Survey Report included in House Document #795 contains the following information (page 27). "Among the inheritances received by the United States from the canal company was the operation of the Bournedale Ferry, about a mile and a quarter west of the Sagamore Bridge. The company, when arranging for the right of way, found that the canal would cut across a local road leading to the Bournedale railroad station (now abandoned) and vicinity, and used principally by persons of the immediate neighborhood. … The company was accordingly obliged to establish and operate a free ferry, and under the general terms of the agreement for purchase, the United States assumed the obligation." The ferry had carried more than 4,700 passengers in 1929, down from 34,800 in 1919. The survey report concluded that accommodation of foot traffic serviced by the ferry could be met by providing for such in the planned centrally located high-level highway bridge or by a walkway across the lock to be built near Bournedale.

The 1930 Survey Report included in House Document #795 also discusses the Government's obligation to provide a railroad bridge (page 90), and the assumption of the responsibilities of the Canal Company for the construction and maintenance of the railroad bridge and its lighting and signaling. The report states that "the assumption of these obligations by the United States as a part of its purchase of the canal was approved by the Chief of Engineers on May 4, 1928, thus the upkeep and operation of the bridge and of a portion of its appertaining signal system are paid for by the United States …"

The Annual Report of the Chief of Engineers for 1933 states that "the construction of bridges over the canal and widening as recommended in House Document #795 … have been included in the Public Works Program (Federal Emergency Administration of Public Works) under the National Industrial Recovery Act appropriations for Fiscal Year 1934." The National Industrial Recovery Act, 73rd Congress, 1st Session, 16 June 1933 (P.L. 73-67) declared the financial situation to be a national emergency, and was enacted to "encourage national industrial recovery, foster fair competition and for construction of certain public works." Much of Title I of this Act was later ruled unconstitutional by the US Supreme Court (May 1935). Title II authorized the President to create new agencies, specifically the Federal Emergency Administration of Public Works (the Public Works Administration). The PWA and its appropriations were used to fund a wide range of programs and projects, including construction of river and harbor improvements and flood control projects, and for military purposes. The PWA would be used to initially authorize improvements to the Cape Cod Canal, including the three new bridges and the deepening and widening of the channel.

That Annual Report for 1933 also states that the Bournedale Ferry service was discontinued on 15 August 1932. The Annual Report for 1934 states that construction of two high-level four-lane highway bridges commenced on 8 December 1933, with construction of the new vertical lift railroad bridge beginning on 18 December 1933. The annual reports for these and the next several years separately account for improvement work done for the Cape Cod Canal with PWA funds, and regular Civil Works funds, as well as civil work operations and maintenance work.

House Committee on Rivers and Harbors Document #15, 74th Congress, 1st Session, 26 December 1934, prints a report of the Chief of Engineers dated 26 December 1934, a report of the BERH dated 10 December 1934, and reports of the Division and District Engineers dated 19 November and 24 October 1934, respectively, on a review of the recommendations made in 1931 in House Document #795. The report recommended eliminating the tidal lock to allow for a sea level canal with a channel depth of -32 feet, 700 Feet Wide from Deep Water in Buzzards Bay to Wings Neck, then 500 Feet Wide Inward from Wings Neck and 540 Feet Wide through the Land Cut with Stone Revetments, two mooring basins; one 2,000 feet long along the north bank near the east entrance, and the other 1,000 feet long in Buzzards Bay north of Hog Island along the southeast channel limit. Also recommended was a small boat harbor at Onset Bay consisting of a channel -15 feet MLW by 100 feet wide by 4,340 feet long from the canal channel into the Bay. This report also states that "the obligations imposed on the United States in acquiring the canal prevented the substitution of a single highway bridge for the two present crossing and two fixed highway bridges are therefore being constructed with a clear span of 550 feet and a vertical clearance of 135 feet above high water. A new railroad bridge with a vertical lift of 500 feet span, affording a clearance of 135 feet above high water is also being constructed." The estimate for future operation and maintenance of the canal included in these reports and recommendations included maintenance of the bridges then under construction.

The Emergency Relief Appropriations Act, 74th Congress, 1st Session, 8 April 1935 was passed as Joint House and Senate Resolution making appropriations for emergency relief purposes. This New Deal legislation transferred direct relief efforts from the Federal Government to the states and local governments and appropriated $4.88 billion to fund the Public Works Administration. No projects were specifically named, and funds were allocated to a wide range of projects and programs, including highways, bridges and rivers and harbors projects. This appropriation was the source of funds for constructing the Cape Cod Canal high-level highway bridges and the new railroad lift span, and beginning dredging to widen (to 205 feet) and deepen the Canal channel, relocate and straighten the Buzzards Bay approach channel, and provide additional rip-rap bank protection in the land cut.

The improvements recommended in HCR&H Document #15 were authorized by the River & Harbor Act of 30 August 1935, 74th Congress, 1st Session (P.L. 74-409). The recommendation was for "an open canal 32 feet deep, 540 feet wide in the land cut, 500 feet wide in the new straight channel to Wings Neck, and 700 feet wide beyond Wings Neck, a 15-foot channel into Onset Bay 100 feet wide, mooring basins at each end of the canal at locations and dimensions to be determined by the Chief of Engineers, all at an estimated cost of $25,875,000 (excluding cost of new bridges and widening from 170 to 205 feet) with $400,000 annually for operation, care and maintenance, which shall include maintenance of the new bridges now under construction." Construction of these improvements, some of which were already underway in 1935 using PWA funds appropriated in Fiscal Year 1934 by the NIRA Act, would be completed in 1940. The mooring basin sizes were further modified during construction with final dimensions as follows: East Basin - 2,500 feet long by 350 feet wide by -25 feet MLW, West Basin - 3,300 feet long by 350 feet wide by -32 feet MLW. Work of removing the old draw span highway bridges began with the old Sagamore Bridge in June 1935 after completion of the new bridge and its approach roads. The removal of all three old bridges and their piers was completed by July 1936.

The Annual Report for 1937 states that "by date of 1 July 1935, under Authority of the Permanent Appropriations Repeal Act of 26 June 1934, operation and maintenance of the Canal were included in the authorized project." The text of the 1934 PAR Act specifically speaks to the Cape Cod Canal only in terms of including the payment of the Canal bonds now being subject to annual appropriations action by Congress, instead of continuing appropriations from the general fund of the Treasury. However, also included in the Act was language that required "operating and care of canals and other works of navigation", also be subject to specific annual appropriations.

The 15-foot Onset Bay small craft channel was initially completed in Fiscal Year 1937. The outer end of the channel was realigned in May to June 1940. In May to June 1957 the Onset Bay channel was extended to the Town Wharf where a 15-foot turning basin and 8-foot anchorage were also dredged, as recommended in House Document #431, 77th Congress, 1st Session, 7 November 1941, and as authorized by the River and Harbor Act of 2 March 1945 (P.L. 79-14).

Improvement dredging of the -13-foot MLW outer section of the East Boat Basin at Sandwich, and the 18-foot West Boat Basin at Bourne, was accomplished between August 1938 and March

1939. Expansion of the East Boat Basin by adding an inner 4.3-acre by -8-foot MLW area was dredged in July 1962 to April 1963, as recommended in House Document #168, 85th Congress, 1st Session, 29 April 1957, and as authorized by the River & Harbor Act of 3 July 1958.

The first repainting and resurfacing of the highway bridges was carried out in the summer of 1938. The first repainting of the Buzzards Bay Railroad Bridge was carried out in May to July 1940.

In summary, as pertains to the highway crossings of the Cape Cod Canal, the Corps has the authority to operate and maintain two highway bridges at Sagamore and Bourne villages, of four travel lanes each, with pedestrian access, and with suitable connection over Federal lands to approaches and highways.

Tables showing the authorization history, and the construction and maintenance history, for the Cape Cod Canal Federal Navigation Project and its associated small boat harbors follow.

CAPE COD CANAL BOURNE, WAREHAM & SANDWICH, MASSACHUSETTS LIST OF AUTHORIZATIONS		
Authorization	**Work Authorized & Constructed**	**Construction**
25 July 1918	US Railroad Administration began to Operate the Canal, Pending Owner's Bankruptcy and Dredged Shoals to Restore the -25 Foot Depth	Federal Take-over of Canal Operations
River & Harbor Act of 21 January 1927	Purchase from the Boston, Cape Cod and New York Canal Company Authorized – At Purchase Canal had Dimensions of -25 Feet MLW by 100 Feet Wide through Land Cut. Design of Bank Revetments began. Possession Taken 31 March 1928	Federal Purchase of Canal Project First US Maintenance August 1928
Public Works Administration Program in the National Industrial Recovery Act of 6 September 1933	Widen Canal Land Cut to 205 Feet and Construct 3 Bridges – A Railroad Bridge – 544 Foot Long Single Track Vertical Lift Span with Closed Vertical Clearances of +7 Feet mhw and 135 Feet MHW Raised, and Horizontal Clearance of 500 Feet and Two 4-Lane High Level Fixed Span Highway Bridges with Vertical Clearance of 135 Feet mhw and 500-Foot Horizontal Clearance.	Land Cut Widening: Oct 1932 - March 1936 Railroad Bridge: Dec 1933 – Dec 1936 Highway Bridges: Dec 1933 - 1935
Permanent Appropriations Repeal Act of 26 June 1934	Authorizes Future O&M Activities on Improvements Authorized by Corps Legislation Only (Also Chief of Engineer's Letter, 1 July 1935)	Future Maintenance Authorized

National Industrial Recovery Act of 6 September 1933	Channel -30 Feet MLW by 500 Feet Wide in Seward Portions and 205 Feet Wide in the Land Cut, the Channel through Buzzards Bay to Follow a Straightened Route Westerly of Mashnee Island with an Increased Width of 700 Feet Beyond Wings Neck, with a Channel -15 Feet MLW into Central Onset Bay for Small Craft Refuge and with 150 Feet Vertical Clearances for the 2 Highway Bridges and a Level Grade Vertical Lift Span Railroad Bridge.	Authorization was Superseded by the Two 1935 Acts
Emergency Relief Act of 28 May 1935	Authorized Dredging and Bank Protection Measures	See Next Entry
River & Harbor Act of 30 August 1935 and Emergency Relief Appropriations Act of 8 April 1935	(1) Eliminating the Tidal Lock from the Authorized Design and Substituting an Open Sea Level Canal -32 Feet MLW by 700 Feet Wide from Deep Water in Buzzards Bay to Wings Neck, then 500 Feet Wide Inward from Wings Neck and 540 Feet Wide through the Land Cut with Stone Revetments and (2) a Channel -15 Feet MLW by 100 Feet Wide by 4,340 Feet Long into Central Onset Bay to Provide a Harbor of Refuge for Small Craft, (3) Two Mooring Basins, One along the North Bank Near the East Entrance 2,500 Feet Long by 350 Feet Wide by -25 Feet MLW and the other in Buzzards Bay North of Hog Island along the Southeast Channel Limit 3,300 Feet Long by 350 Feet Wide by -32 Feet MLW, (4) Accessory Facilities and Features Including Lighting, Aids to Navigation and Operations Facilities, (5) Two Fixed Span Highway Bridges Each with a 150 Foot Vertical Clearance, and (6) a Level Grade Vertical Lift Span Railroad Bridge.	32-Foot Channel: Aug 1935 – Nov 1939 Onset Bay Channel: July 1936 – FY 1937 18-Foot West Boat Basin: Aug 1938 – March 1939
Public Works Administration, 29 April 1935, under the Authority of the National Industrial Recovery Act of 6 September 1933	Construction of the East Boat (Mooring) Basin	East Boat Basin: Aug 1938 – March 1939

River & Harbor Act of 2 March 1945	Onset Bay - Extend the 15-Foot Channel 3,560 LF Upstream 150 Feet Wide to a 15-Foot 7.1 Acre Turning Basin (460 by 550 Feet) at the Town Wharf about, and Two 8-Foot Anchorage Areas - 5.1 Acres East of the Basin and 10.2 Acres South of the Basin and West of the Channel	May 1957 – June 1959
River & Harbor Act of 30 June 1948	Buttermilk Bay - Channel -7 Feet MLW by 100 Feet Wide Across the Outer Bar from off Taylor Point to Sears Point, Widened at the Bend	Nov 1952 – Jan 1953
River & Harbor Act of 3 July 1958	Expansion of the East Boat Basin to a Total Area of 7 Acres by Adding 4.3 Acres at -8 Feet MLW	July 1962 – April 1963
Water Resources Development Act of 1986, Section 1002	Deauthorized Raising the Inshore End of the South Jetty at the East Entrance to the Canal as Authorized by the River & Harbor Act of 1960 as Part of the Town Neck Beach Erosion Control Project	Deauthorization
Water Resources Development Act of 8 November 2007, P.L. 110-114, §1004(a)(8)	Directed a Study, and if Found Feasible, Implementation of Improvements to the East Boat Basin, Cape Cod Canal, under Section 107 Authority	Never Acted On – Study Terminated at Sponsor Request
America's Water Infrastructure Act, 23 Oct 2018 (PL 115-270) 132 Stat. 3765 §1315	States that the Secretary May Repair or Replace as Necessary, any Bridge Owned or Operated by the Secretary that is in New England and Necessary for Evacuation during an Extreme Weather Event	

CAPE COD CANAL, BOURNE, WAREHAM & SANDWICH PROJECT CONSTRUCTION & MAINTENANCE HISTORY		
Work Dates	**Work Accomplished**	**Quantities**
1918 - 1927	US Railroad Administration has Operated the Canal Since 25 July 1918 and has Dredged Shoals to Restore the -25 Foot Depth.	Unknown
August 1928	Maintenance Dredging of 25-Foot Land Cut West of Sagamore Bridge	7,795 cy
Sept 1928 – FY30	Maintenance Dredging of 25-Foot Channel by US Dredge *Minquas* beginning at the East Entrance and Proceeding West	637,156 cy
Nov 1929 – Feb 1930	Maintenance Dredging of 25-Foot Buzzards Bay Approach Channel by US Dredge *Marshall*	340,177 cy
FY 1931 – FY 1932	Maintenance Dredging of 25-Foot Channel by US Dredge *Minquas*	630,450 cy
FY 1931 – FY 1932	Placement of Riprap Bank Protection along Land Cut Slopes	29,118 Tons Stone
July 1932 – Sept 1932	Construction of an 18-Inch Concrete Drain on the South Bank at the East End of the Canal	NA
Aug 1932 – Sept 1932	Maintenance Dredging of Hard Shoals at the Eastern End of the Canal	12,099 cy Plus 27 cy Boulders
FY 1933	Maintenance Dredging of 25-Foot Channel by US Dredge *Minquas*	396,790 cy
Oct 1932 – March 1933	Begin Improvement Dredging to Widen 25-Foot Channel Land Cut from the Eastern Entrance Proceeding Westerly	660,244 cy Plus 1,763 cy Boulders
June 1933 – Dec 1933	Continue Improvement Dredging to Widen 25-Foot Channel Land Cut	65,742 cy Plus 3,852 cy Boulders
FY 1933	Placement of Stone Paving along Canal Banks	3,628 sf Stone
Dec 1933 – Dec 1935	Begin Construction of Bourne Bridge Piers & Abutments and Highway Approaches	Unknown
Dec 1933 – June 1935	Begin Construction of Sagamore Bridge Piers & Abutments and Highway Approaches	Unknown

Dec 1933 – Dec 1935	Begin Construction of Railroad Bridge Piers and Abutments	Unknown
FY 1934	Maintenance Dredging of 25-Foot Channel by US Dredge *Minquas*	384,047 cy
May 1934 – Aug 1934	Placement of Stone Paving along Canal Banks	3,000 Tons Stone
June 1934 – Dec 1934	Continue Improvement Dredging to Widen 25-Foot Channel Land Cut	1,002,044 cy Plus 5,408 cy Boulders
May 1934 – Oct 1935	Beginning Construction of the Approaches and Superstructures of the Bourne and Sagamore Highway Bridges	N/A
July 1934 – Aug 1934	Maintenance Dredging of 25-Foot Channel by US Dredge *Minquas* & *Marshall*	534,881 cy
Oct 1934 – April 1935	Removal of Boulders from the Easterly Approach Channel in Buzzards Bay by US Lighter	Unknown
Oct 1934 – June 1935	Install Lighting Systems on Highway Bridges	N/A
Nov 1934 – Dec 1935	Complete Construction of the Superstructure (Towers & Span) of the Railroad Bridge	NA
March 1935 – July 1935	Relocation of the Cape Shore Highway	N/A
April 1935	Improvement Dredging to Widen the 25-Foot Channel Cut through Buzzards Bay	111,381 cy
May 1935 – June 1935	Maintenance Dredging of 25-Foot Channel by US Dredge *Comstock*	303,125 cy
June 1935 – Sept 1935	Demolition of Old Highway Draw Bridges	N/A
June 1935 – Oct 1935	Improvement Dredging to Expand the 25-Foot East Mooring Basin	1,378,391 cy
July 1935 – June 1936	Maintenance Dredging of 25-Foot Channel by US Dredge *Minquas*	628,163 cy
Nov 1935 – March 1936	Improvement Dredging to Widen Channel Cut at Site of Old Bridge Piers and Removal of Old Piers	193,008 cy Plus 628 cy Boulders & 2,351 cy Old Concrete Piers

Dec 1935 – May 1936	Improvement Dredging to Widen the 25-Foot Channel Land Cut	1,202,359 cy Plus 5,238 cy Boulders
Nov 1935 – Dec 1936	Relocation of Southern Approach Railroad	NA
FY 1936 – FY 1937	Placement of Stone Paving along Canal Banks	Unknown
Jan 1936 – May 1936	Demolition of Old Railroad Draw Span	NA
Aug 1935 – Feb 1937	Improvement Dredging of 32-Foot Channel in Center Cut in Land Cut and Hog Island Channel Reaches with Disposal to Construct Stony Point Dike	8,359,936 cy 1,417 cy Boulders
July 1936	Removal of Remains of Old Concrete Highway Bridge Pier at Sagamore	Unknown
July 1936 – FY 1937	Improvement Dredging of 17-Foot Onset Bay Channel	Unknown
July 1936 – April 1937	Maintenance Dredging of 32-Foot Canal Land Cut in Reaches and Widths already Finished by U.S. Hopper Dredge *Minquas*	520,424 cy
Aug 1936 – Feb 1937	Construction of Steel Sheet Pile Bulkhead along South Bank at East End of Canal	????
July 1936 – May 1938	Maintenance and Improvement Dredging to Widen at 25 Feet and Deepen to 32 Feet in Canal Land Cut at East End	3,545,280 cy Plus 10,354 cy Boulders
July 1936 – Aug 1938	Continue Improvement Dredging of 32-Foot Canal Land Cut, Hog Island Channel Reaches, and West Mooring Basin, and Removal of Additional Old Bridge Piers	7,887,248 cy Plus 18,124 cy Boulders
Feb 1937 – Feb 1938	Improvement Dredging to Widen 32-Foot Cut at East and West Ends of the Canal Land Cut by U.S. Hopper Dredge *Marshall*	1,776,621 cy
Sept 1937 – Nov 1937	Excavation and Placement of Revetment, Drains, Culverts, Roads, etc	Unknown
July 1937 – May 1938	Maintenance Dredging of 32-Foot Canal Land Cut in Reaches and Widths already Finished by U.S. Hopper Dredge *Marshall*	286,136 cy

January 1938	Maintenance Dredging of 25-Foot East Mooring Basin	184,300 cy
Oct 1938 – March 1940	Repairs to Revetment on Canal Banks Damaged by Hurricane of Sept 1938	Unknown
Dec 1938 – June 1939	Maintenance Dredging of 32-Foot Canal Reaches and 15-Foot Onset Bay Channel	304,767 cy
Dec 1938 – June 1939	Maintenance Dredging of 15-Foot Onset Bay Channel	Unknown
June 1939 – Sept 1940	Construction of Revetment on Sandy Point Dike	Unknown
Aug 1938 – March 1939	Improvement Dredging of 32-Foot Canal Land Cut, 13-Foot East Boat Basin and 18-Foot West Boat Basin	570,892 cy Plus 3,342 cy Boulders
June 1938 – Oct 1938	Maintenance Dredging of 32-Foot Canal Reaches	425,019 cy
Sept 1938 – Dec 1938	Continue Improvement Dredging of 32-Foot Hog Island Channel Reach and 32-Foot West Mooring Basin	1,087,093 cy
Nov 1938 – June 1940	Continue Improvement Dredging of 32-Foot Canal Land Cut	3,392,163 cy Plus 22,033 cy Boulders
July 1939 – Jan 1941	Maintenance Dredging of 32-Foot Canal Reaches by U.S. Hopper Dredges *Atlantic*, *Minquas* & *Marshall*	3,583,784 cy
Oct 1939 – Nov 1939	Continue Improvement Dredging of 32-Foot Cleveland Ledge Channel Reach	Unknown
March 1940 – April 1940	Excavation and Placement of Revetment on Canal Banks on North Side and around the West Boat Basin	Unknown
May 1940 – June 1940	Improvement Dredging to Relocate Outer Alignment of 15-Foot Onset Bay Channel	21,492 cy Plus 129 cy Boulders
July 1940	Maintenance Dredging of 32-Foot Canal Reaches by U.S. Hopper Dredge *Minquas*	19,180 cy
July 1940 – May 1941	Removal of Boulders from the Land Cut & Cleveland Ledge Reach	2,298 cy Boulders

Aug 1940 – Oct 1940	Construction of Steel Mooring Dolphins at West Boat Basin	NA
Sept 1940 – Dec 1940	Construction of Riprap Slope Protection around the East Boat Basin	Unknown
Apr 1941 – May 1941	Planting Beach Grass on Stony Point Dike	NA
May 1941 – June 1941	Hydraulic Maintenance Dredging of the 25-Foot East Mooring Basin	192,509 cy
FY 1942	Blasting and Removal of Large Boulders from the 32-Foot Channel	946 cy Boulders
FY 1942	Maintenance of Riprap South Slope of Land Cut with Crushed Stone (Sta. 49 to 51)	Unknown
July 1941 – Aug 1941	Maintenance Dredging of the 32-Foot Hog Island Channel Reach by U.S. Hopper Dredge *Absecon*	57,750 cy
Feb 1942 – March 1943	Maintenance Dredging of the 32-Foot Channel, Approaches and Mooring Basins by U.S. Hopper Dredge *Atlantic*	1,623,737 cy
FY 1943	Continue Blasting and Removal of Large Boulders from the 32-Foot Channel	522 cy Boulders
July 1942 – FY 1943	Repairs to the Canal Slope Revetments, Eroded and as the Result of a Wreck in the Canal	Unknown
September 1942	Hydraulic Dredging to Place Material on Eroded Sections of Sandy Point Dike	Unknown
Nov 1942 – Jan 1943	Removal of Temporary Pier and Dolphins from Sandy Point Dike	NA
July 1943 – Oct 1943	Maintenance Dredging of the 32-Foot Canal Channel by U.S. Hopper Dredge *Marshall*	506,343 cy
FY 1944	Continue Blasting and Removal of Large Boulders from the 32-Foot Channel	278 cy Boulders
Sept 1943 – Nov 1943	Road Grading and Riprap Placement on Stony Point Dike	Unknown
January 1944	Maintenance Dredging of the 32-Foot Hog Island Channel by U.S. Hopper Dredge *Atlantic*	150,410 cy
Sept 1944 – Nov 1944	Improvement Dredging to Widen the 32-Foot Channel through the Hog Island Reach	170,780 cy Plus 779 cy Boulders

FY 1945	Continue Blasting and Removal of Large Boulders from the 32-Foot Channel	206 cy Boulders
April 1945 – May 1945	Maintenance Dredging of the 32-Foot Channel, Approaches and West Mooring Basin by U.S. Hopper Dredge *Atlantic*	549,349 cy
June 1945 – Aug 1945	Maintenance Dredging of the 25-Foot East Mooring Basin	93,041 cy
Sept 1945 – Oct 1945 Apr 1946 – May 1946	Maintenance Dredging of the 32-Foot Channel by U.S. Hopper Dredge *Atlantic*	295,076 cy
Oct 1945 – Dec 1945	Maintenance of Slopes of Land Cut with Crushed Stone and Gravel	30,950 cy Gravel Placed
May 1946 – Aug 1946	Depositing Gravel on Eroded Sections of the North and South Banks of the Canal	69,247 cy Gravel Placed
FY 1946	Continue Blasting and Removal of Large Boulders from the 32-Foot Channel	175 cy Boulders
May 1947 – June 1947	Maintenance Dredging of the 32-Foot Channel by U.S. Hopper Dredge *Atlantic*	182,679 cy
FY 1947	Continue Blasting and Removal of Large Boulders from the 32-Foot Channel	25 cy Boulders
October 1947	Maintenance Dredging of the 32-Foot Channel by U.S. Hopper Dredge *Atlantic*	64,466 cy
May 1948 – Aug 1948	Maintenance Dredging of the 32-Foot Channel by U.S. Hopper Dredge *Atlantic* and *Lyman*	847,690 cy
FY 1949	Emergency Agitation Dredging to Removal a Shoal from the 32-Foot Canal Channel	Unknown
June 1950 – Aug 1950	Maintenance Dredging of the 32-Foot Channel by U.S. Hopper Dredge *Lyman*	387,610 cy
May 1952	Raising of the Wreck of the *MS Arizona Sword* from the East End of the Canal Channel	Wreck Removal
Oct 1951 – Aug 1952	Maintenance Dredging of the 32-Foot Channel by U.S. Hopper Dredge *Lyman*	506,637 cy
April 1953 – May 1953	Maintenance Dredging of the 32-Foot Channel by U.S. Hopper Dredge *Goethals*	524,356 cy

Nov 1953 – July 1954	Maintenance Dredging of the Canal Land Cut, Hog Island Channel and Cleveland Ledge Channel by U.S. Hopper Dredge *Goethals*	600,610 cy
Nov 1954 – Dec 1954	Maintenance Dredging of the 32-Foot Channel by U.S. Hopper Dredge *Gerig*	315,800 cy
Nov 1955 – Dec 1955	Maintenance Dredging of the 32-Foot Channel in the Cleveland Ledge and Hog Island Reaches and the West Mooring Basin by U.S. Hopper Dredge *Comber*	186,284 cy
August 1956	Maintenance Dredging of the 32-Foot Channel by U.S. Hopper Dredge *Comber*	418,086 cy
April 1957	Maintenance Dredging of the 32-Foot Channel by U.S. Hopper Dredge *Comber*	165,042 cy
May 1957 – June 1957	Onset Bay - Improvement Dredging of the 15-Foot Channel, Turning Basin and Two 8-Foot Anchorage Areas	175,000 cy
April 1958	Maintenance Dredging of Shoals in the 32-Foot Canal Land Cut, Cleveland Ledge and Hog Island Reaches by U.S. Hopper Dredge *Comber*	266,970 cy
June 1958	Onset Bay – Removal of a Large Boulder from the 15-Foot Channel	One Boulder
May 1959	Maintenance Dredging of Shoals in the 32-Foot Canal Land Cut, Cleveland Ledge and Hog Island Reaches by U.S. Hopper Dredge *Goethals*	176,440 cy
May 1959 – June 1959	Onset Bay – Improvement - Removal Rock and Hard Material from the 15-Foot Channel and 8-Foot Anchorage	Unknown
May 1960	Maintenance Dredging of Shoals in the 32-Foot Canal Land Cut, Cleveland Ledge and Hog Island Reaches by U.S. Hopper Dredge *Comber*	99,150 cy
July 1960	Maintenance Dredging of the West Mooring Basin to –7 Feet	8,710 cy
Sept 1959 – Oct 1959	Blasting and Removal of Large Boulders and Hard Shoal Areas in the 32-Foot Canal	4,640 cy Hard Material and Boulders

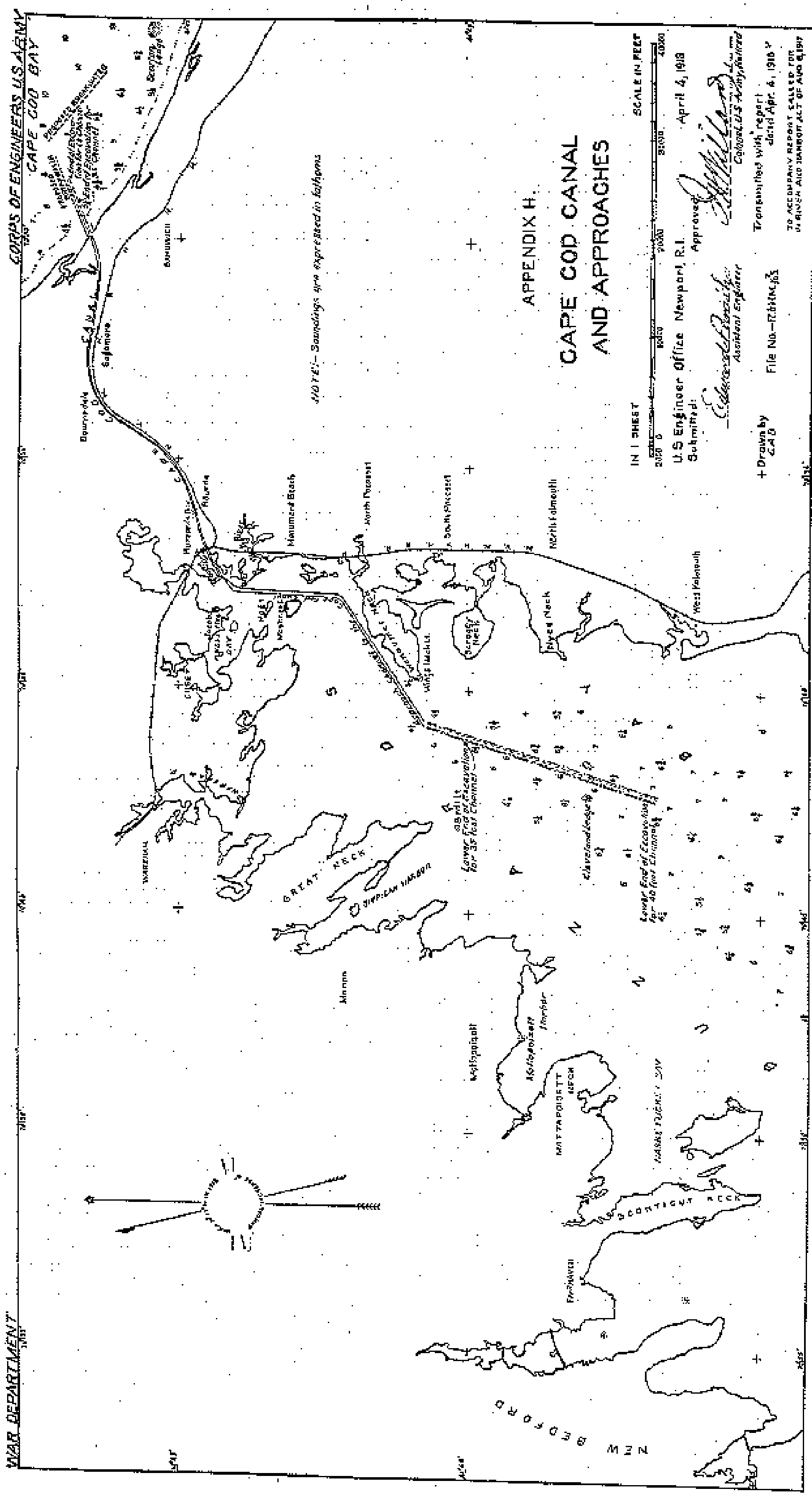
May 1961	Maintenance Dredging of Shoals in the 32-Foot Canal Land Cut, Cleveland Ledge and Hog Island Reaches by U.S. Hopper Dredge *Comber*	343,650 cy
March 1961 – Oct 1961	Improvement Dredging and Dry Excavation for Widening of the Hog Island Channel Reach along the SE Limit	260,786 cy
April 1961 – June 1961	Removal of Boulder Shoals from the Cleveland Ledge and Hog Island Channel Reaches	241 cy Boulders
May 1962 – June 1962	Maintenance Dredging of Shoals in the 32-Foot Canal Land Cut, Cleveland Ledge and Hog Island Reaches by U.S. Hopper Dredge *Comber*	297,897 cy
July 1962 – April 1963	Improvement Dredging to Expand the East Boat Basin by Adding the 8-Foot by 4.3-Acre Anchorage	192,000 cy
Sept 1962 – Oct 1963	Repairs to the North Jetty (Breakwater) at East Entrance to the Canal	27,700 Tons Stone
May 1963	Maintenance Dredging of Shoals in the 32-Foot Canal Land Cut, Cleveland Ledge and Hog Island Reaches by U.S. Hopper Dredge *Comber*	102,820 cy
March 1963 – April 1963	Blasting and Removal of Large Boulders from the Hog Island Channel Reach	65 cy Boulders Estimated
Feb 1964 – May 1964	Repairs to Riprap Slope Protection along Canal Land Cut Banks	6,000 Tons Stone Estimated
May 1964	Maintenance Dredging of the 32-Foot Canal Land Cut, Cleveland Ledge and Hog Island Reaches by U.S. Hopper Dredge *Goethals*	100,390 cy
May 1965 – June 1965	Maintenance Dredging of the 32-Foot Canal Land Cut, Cleveland Ledge and Hog Island Reaches by U.S. Hopper Dredge *Goethals*	137,900 cy
May 1965 – Jan 1966	Blasting and Removal of Large Boulders from the Cleveland Ledge Channel Reach	300 cy Boulders Estimated
May 1965 – May 1966	Repairs to Riprap Slope Protection along Canal Land Cut Banks	11,675 Tons Stone
April 1966 – May 1966	Maintenance Dredging of the 32-Foot Canal Land Cut, Cleveland Ledge and Hog Island Reaches by U.S. Hopper Dredge *Goethals*	84,500 cy

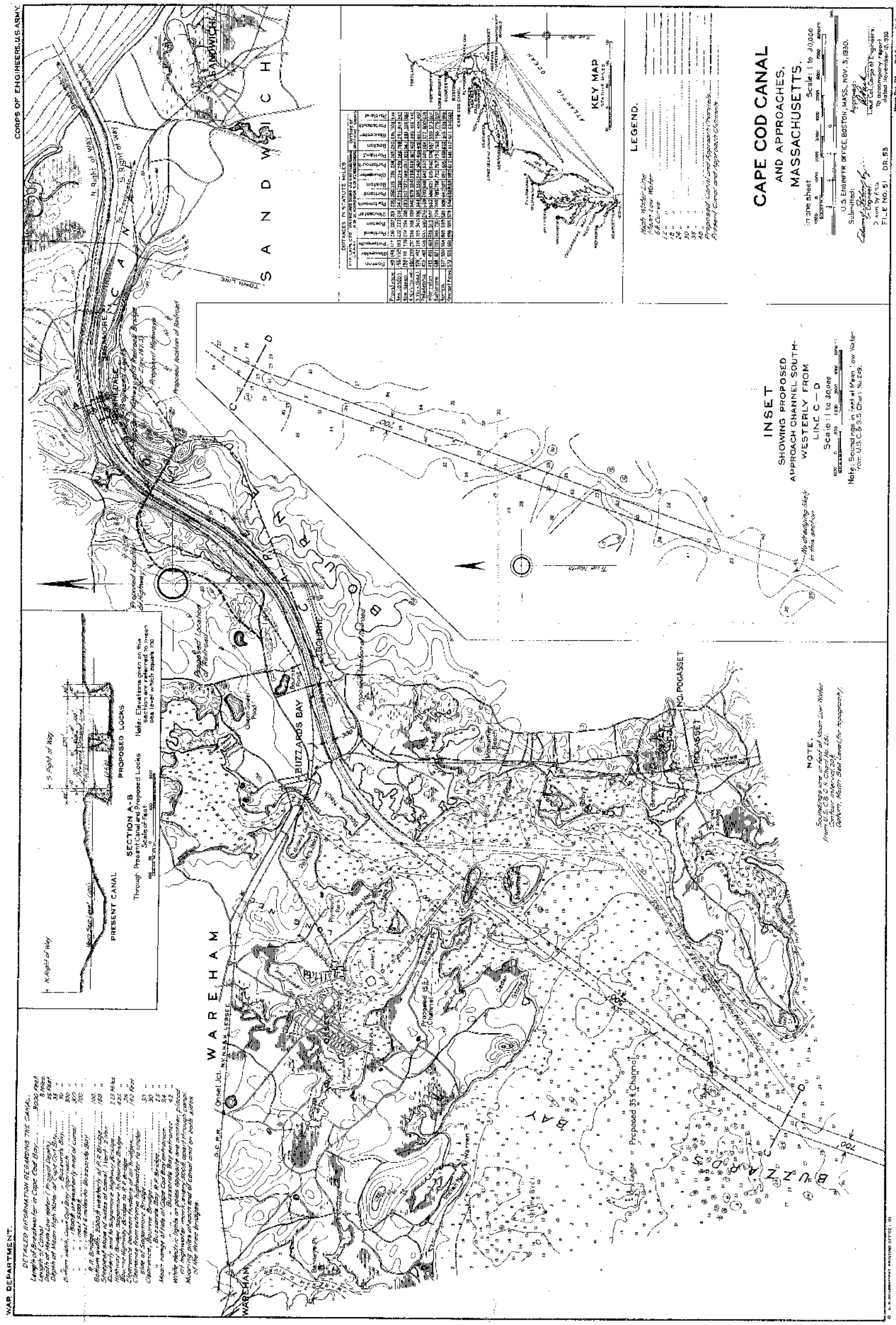
June 1967 – July 1967	Maintenance Dredging of Shoals in the 32-Foot Canal Land Cut, Cleveland Ledge and Hog Island Reaches by U.S. Hopper Dredge *Comber*	35,584 cy 25,135
March 1967 – June 1967	Repairs to Riprap Slope Protection along Canal Land Cut Banks	5,836 Tons Stone
June 1968	Maintenance Dredging of Shoals in the 32-Foot Canal Land Cut, Cleveland Ledge and Hog Island Reaches by U.S. Hopper Dredge *Comber*	136,000 cy
FY 1968 - May 1968	Repairs to Riprap Slope Protection along Canal Land Cut Banks	2,245 Tons Stone
FY 1969	Removal of Boulders from the Canal Channels	Unknown
March 1969 -	Repairs to the Steel Sheet-Pile Bulkhead at the East Boat Basin	NA
Nov 1969 – Feb 1970	Repairs to Riprap Slope Protection along Canal Land Cut Banks	Unknown
June 1970	Maintenance Dredging of Shoals in the 32-Foot Canal Land Cut, Cleveland Ledge and Hog Island Reaches by U.S. Hopper Dredge *Comber*	154,372 cy
FY 1972	Repairs to Riprap Slope Protection along Canal Land Cut Banks	Unknown
FY 1973	Installation of Steel Mooring Dolphins	Unknown
June 1973	Maintenance Dredging of Shoals in the 32-Foot Canal Channel	Unknown
July 1973 – Aug 1973	Maintenance Dredging of Shoals in the 32-Foot Canal Channel by U.S. Hopper Dredge	100,000 cy
FY 1974	Installation and Repair of Steel Mooring Dolphins at the East and West Mooring Basins, and Repairs to Riprap Slope Protection	Unknown
June 1975	Maintenance Dredging of Shoals in the 32-Foot Canal Channel by U.S. Hopper Dredge	125,620 cy
June 1975	Maintenance Dredging of the West Boat Basin	4,900 cy
FY 1975	Removal of a Sunken Vessel from the East Boat Basin	Wreck Removal
FY 1975	Rehabilitation of the South Jetty at the East Entrance to the Canal	15,500 Tons Stone, Est.

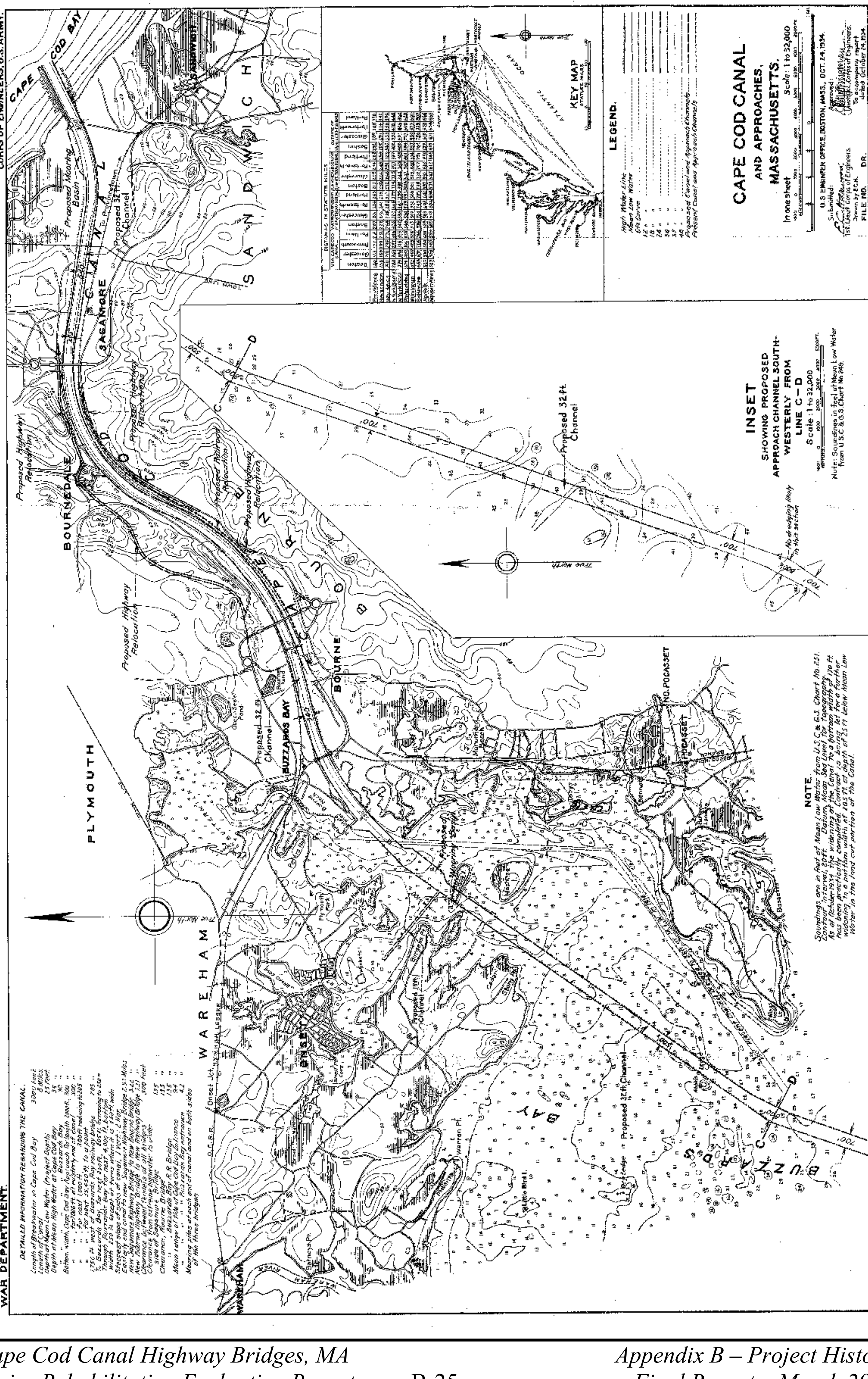
FY 1977	Maintenance Dredging of Shoals in the 32-Foot Canal Channel by U.S. Hopper Dredge	73,054 cy
FY 1980	Removal of a Boulder from the 32-Ft Channel	One Boulder
FY 1981	Removal of the Sunken Vessel *Mary J. Landry*	Wreck Removal
FY 1982	Replacement of Docks, Pilings and Dolphins in the West Boat Basin	Unknown
FY 1984	Installation of a New Electronic Traffic Control System, and Radar System	NA
FY 1985 – Sept 1986	Reconstruction of the Bulkhead along the Cape Shore on Either Side of the Entrance to the East Boat Basin with a Steel Sheet Pile Bulkhead with Associated Fendering System	Unknown
FY 1986	Purchase of Riprap Stone for Future Repairs to Canal Banks	Unknown
FY 1987	Maintenance Dredging of the 32-Foot Canal Channel by U.S. Hopper Dredge *McFarland*	177,000 cy
FY 1988 – March 1989	Repairs to Mooring Dolphins and Marine Railways at the West Boat Basin	NA
March 1990 - May 1990	Maintenance Dredging of the 32-Foot East Mooring Basin	121,952 cy
July 1992 – June 1993	Emergency Shoreline Protection at Wings Neck	Unknown
FY 1999 – FY 2000	Maintenance Dredging of the 32-Foot Canal Channel by Contract Hopper Dredge from Hog Island Reach Easterly, with Material used for CAD Cell Capping at Boston Harbor	162,200 cy
Sept 1999 – Oct 2000	Repairs to the South Jetty at the East Entrance to the Canal	Unknown
May 2000 – Aug 2002	Construction of Salt Marsh Restoration Project at Sagamore Marsh including Tidal Flow Culvert with Gates and Dike	NA
Sept 2000 – Feb 2002	Repairs to Docks and Mooring Dolphins	NA
April 2001 – FY03	Major Rehabilitation of the Buzzards Bay Railroad Bridge	NA

September 2002 – November 2002	Maintenance Dredging of the 32-Foot Channel in the Cleveland Ledge and Hog Island Reaches and Realignment of the Western Approach to Cleveland Ledge	117,000 cy plus 30 cy Boulders and 5 Minor Unquantified Shoals
FY 2008	Minor Repairs to the Canal Bank Revetment	NA
January 2010 – March 2010	Maintenance Dredging of the 32-Foot Channel and 25-Foot East Mooring Basin by Hopper Dredge with Material used to Cap CAD Cells in Boston Harbor. Contractor Over-dredged the Mooring Basin to 32 Feet at Own Expense to Yield Material for the Capping Project.	20,837 CY
Dec 2015 – June 2016	Maintenance Dredging of the East End of the 32-Foot Channel and East Mooring Basin with Placement on Town Neck Beach at Local Cost	118,029 cy

Project Maps from the 1919, 1930 and 1934 House Documents are provided below.


CORPS OF ENGINEERS U.S. ARMY
CAPE COD BAY
WAR DEPARTMENT
APPENDIX H.
CAPE COD CANAL
AND APPROACHES
NOTE:- Soundings are expressed in fathoms
SCALE IN FEET
U.S. Engineer Office Newport, R.I.
April 4, 1918
Submitted:
Approved:
File No.
GREAT NECK
NEW BEDFORD


WAR DEPARTMENT.
CORPS OF ENGINEERS, U.S. ARMY.
CAPE COD CANAL
AND APPROACHES,
MASSACHUSETTS.
INSET
SHOWING PROPOSED
APPROACH CHANNEL SOUTH-
WESTERLY FROM
LINE C – D
KEY MAP
LEGEND.
SECTION A-B
PROPOSED LOCKS
PRESENT CANAL
NOTE.
WAREHAM
SANDWICH
BUZZARDS BAY
BOURNE
SAGAMORE
POCASSET
NO. POCASSET


WAR DEPARTMENT.
CORPS OF ENGINEERS, U.S. ARMY.
CAPE COD CANAL
AND APPROACHES,
MASSACHUSETTS.
U.S. ENGINEER OFFICE, BOSTON, MASS., OCT. 24, 1934.
LEGEND.
KEY MAP
INSET
SHOWING PROPOSED
APPROACH CHANNEL SOUTH-
WESTERLY FROM
LINE C – D
Scale: 1 to 32,000
NOTE.
CAPE COD BAY
BUZZARDS BAY
PLYMOUTH
WAREHAM
SANDWICH
BOURNEDALE
SAGAMORE
BOURNE
NO. POCASSET
POCASSET
ONSET
Proposed 32 Ft. Channel
Proposed Highway Relocation
Proposed Mooring Basin
True North

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