New Hampshire Department of Transportation

IN-DEPTH INSPECTION AND CONDITION REPORT FOR THE PORTSMOUTH MEMORIAL BRIDGE OVER THE PISCATAQUA RIVER

247/084

October 2009

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No. 08820
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Loretta Girard Doughty
10-5-09

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Underwater Inspection Report

MEMORIAL BRIDGE

SUMMARY

The underwater portions of the substructure units were found to be in Fair condition due to the more advanced deterioration to the back channel pier. The concrete on the Piscataqua River piers is sound with no significant deterioration except minor deterioration of the outermost ½-inch around the low water line. No evidence of scour was observed. The limits of the underwater inspection extended from low water to the mudline.

The task also included the extraction of six concrete cores, concrete characterization, and material service life prediction. The findings of that study are contained in a separate report but generally support the visual and tactile findings of the underwater inspection.

INTRODUCTION

In June 2009, Appledore Marine Engineering, Inc. (AMEI) completed an underwater inspection of the Memorial Bridge. The inspection was performed by a four-man dive team under the direction of a Professional Engineer, and included a visual and tactile inspection of the North and South Main Bridge Piers and the Kittery Approach Pier. The other bridge substructure units including the Portsmouth Abutment and Kittery approach column foundations are entirely exposed at low water and therefore were excluded from the scope of the underwater inspection.

Previously, Appledore completed underwater inspections for this structure in 2003 and 2008.

OBJECTIVE

The objective of this project is to provide a general description and assessment with recommendations of the underwater condition of the North and South Main Bridge Piers and the Kittery Approach Pier.

FACILITY DESCRIPTION

The Memorial Bridge was constructed in the early 1920's and is approximately 85 years old. It spans over the Piscataqua River connecting Portsmouth, NH with Kittery, ME. The North and South Main Bridge Piers consist of a concrete foundation placed on bedrock at elevation -83 feet (MLW) and -73 feet, respectively. Both piers are constructed of reinforced concrete and have a granite block fascia protecting the concrete through the tidal zone. A timber fender system encompasses the piers from approximately 2 feet above mean high water down to 5 feet below mean low water. The Kittery Approach Pier is of similar construction with a steel sheetpile cofferdam exposed at low water.

OBSERVATIONS

The structures are generally covered in light marine growth and representative areas were cleaned using hand tools for closer examination. The photos within this report provide a visual representation of the typical underwater conditions and deterioration.

North and South Main Bridge Piers

The concrete is generally in overall sound condition with limited areas of minor deterioration. The deterioration is generally more concentrated around low water. In the low water zone hammer soundings identified that the outer ½-inch of concrete is soft (Photo 1&2). Once the deteriorated concrete was removed, the underlying concrete was sound. The structure also has 12-inch by 12-inch block outs in the concrete from timber members that were originally cast into the piers and marine borers subsequently deteriorated the timber. These voids extend approximately 12-inches into the structure and have had no deleterious effect on the concrete pier (Photo 3).

The timber fender systems have moderate marine borer deterioration throughout (Photo 4). The deterioration has progressed since the 2003 inspection and estimated to have affected approximately

10% - 20% of the timber members. It is important to note that marine borer deterioration is difficult to detect and could be more prevalent within the interior of the timber members.

Kittery Approach Pier

The approach pier is encased in a steel sheet piling. The steel sheet piling has major deterioration due to corrosion and areas of complete section loss. The steel sheeting is believed to be part of the original construction (cofferdam) and likely no longer required, but does provide some abrasion protection for the concrete. No concrete deterioration was observed behind the steel sheeting voids.

ASSESSMENTS

Based on our underwater inspection, the underwater condition of these structures is Satisfactory due to isolated areas of minor deterioration. The deterioration noted in this report is considered minor and no load reductions are required as a result of the underwater structures. Detailed examinations of the concrete cores and service life predictions determined that the concrete below low water has a remaining service life greater than 50 years, provided 1-2 inches of section loss can be tolerated.

The detailed concrete examination and material service life prediction determined that concrete exposed to higher oxygen levels in the tidal and atmospheric zones may require rehabilitation to provide an extended service life.

RECOMMENDATIONS

No repairs are recommended to the underwater portions of the concrete piers.

The fender system is progressively deteriorating and recommended it be replaced in the short term (0-10 years). Our estimate of the probable cost of construction is depicted in Table 1 and is based on in-kind replacement.

TABLE 1

Item No.	Recommended Repairs	Estimated Construction Cost (ECC)
1	Replace fender system in-kind	$ 660,000
	Subtotal	**$ 660,000**
	Est. Engineering fees (15%) Insp., Design, Permits, and Const Admin.	$ 100,000
	TOTAL * say	**$ 760,000**

Costs are in 2009 dollars and include: Contingency, Mobilization, and Contractor Overhead and Profit.

PHOTOGRAPHS

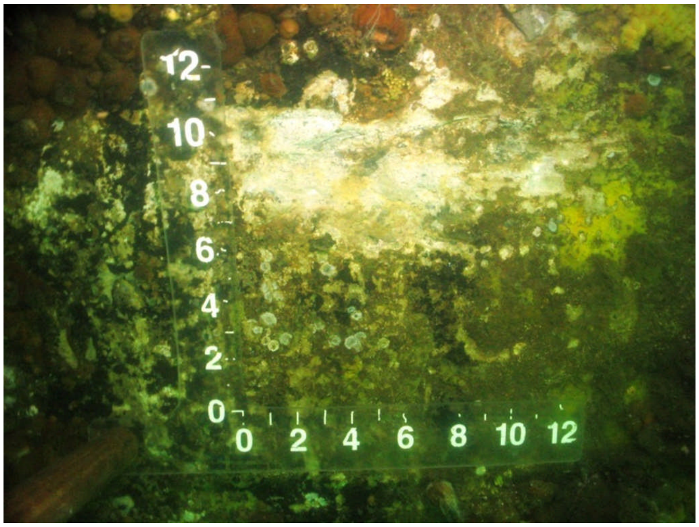


Photo 1:

Typical concrete deterioration around low water. Limited to ½-inch in depth.

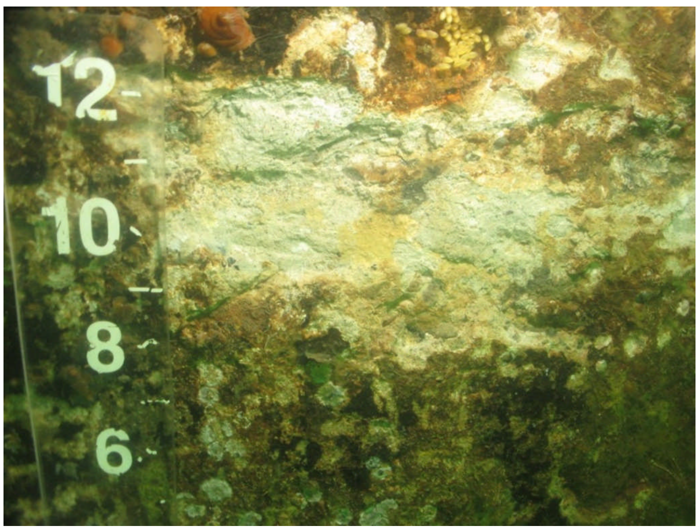


Photo 2:

Close view of deterioration after hammer removal of deteriorated concrete.

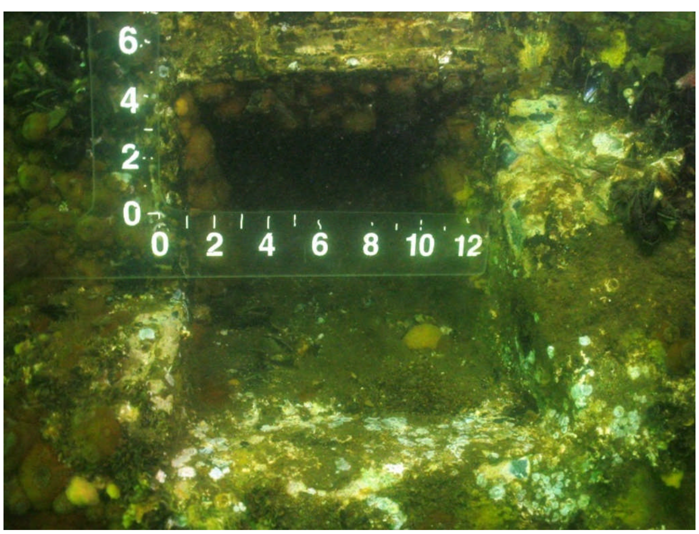


Photo 3:

Typical blockout in the concrete piers where timber elements were once connected. Concrete is sound at these locations.

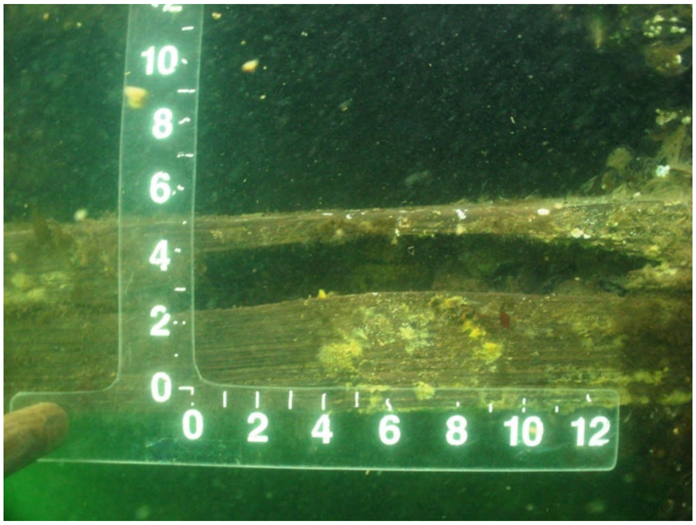
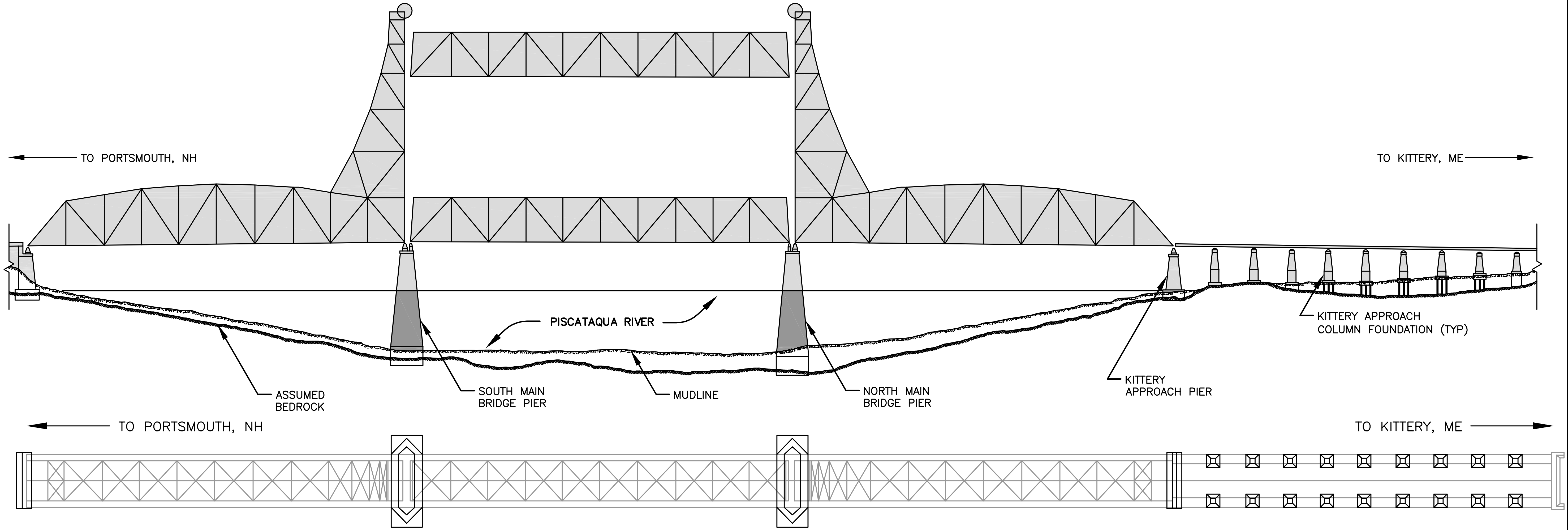


Photo 4:

Marine borer deterioration to the fender system.

FIGURES



CONCRETE CORE SCHEDULE			
CORE I.D.	PIER	ELEMENT	EL. (MLLW)
1–1	NORTH	DOWNSTREAM NOSE	0'
1–2	NORTH	UPSTREAM NOSE	–25'
1–3	NORTH	DOWNSTREAM NOSE	–50'
2–1	SOUTH	UPSTREAM NOSE	0'
2–2	SOUTH	DOWNSTREAM NOSE	–25'
2–3	SOUTH	UPSTREAM NOSE	–50'

LEGEND:

STRUCTURAL UNITS INSPECTED

NOTES:

SCALE 1" = 100'

DRAWING REFERENCE:

STATE OF NEW HAMPSHIRE HIGHWAY DEPARTMENT BRIDGE DIVISION, GENERAL PLAN & ELEVATION. FILE NO. 1–18–3–1, 7/30/43, BRIDGE NO. 2471084

GRAPHIC SCALE	DATE	APPLEDORE MARINE ENGINEERING, INC. PORTSMOUTH, N.H.	HDR ONE COMPANY Many Solutions 695 ATLANTIC AVE | 2ND FLOOR | BOSTON | MA | 02111
AS NOTED	AUGUST 2009	Contract Number:	PORTSMOUTH, NH MEMORIAL BRIDGE PLAN — FIG. NO. 1

5083–Report Memorial Bridge.dwg

CONDITION RATING DESCRIPTIONS

CONDITION RATING DESCRIPTIONS

Rating	Description
Good	No visible damage, or only minor damage is noted. Structural elements may show very minor deterioration, but no overstressing is observed. No repairs are required.
Satisfactory	Limited minor to moderate defects or deterioration are observed, but no overstressing is observed. No repairs are required.
Fair	All primary structural elements are sound, but minor to moderate defects or deterioration is observed. Localized areas of moderate to advanced deterioration may be present but do not significantly reduce the load-bearing capacity of the structure. Repairs are recommended, but the priority of the recommended repairs is low.
Poor	Advanced deterioration or overstressing is observed on widespread portions of the structure, but does not significantly reduce the load-bearing capacity of the structure. Repairs may need to be carried out with moderate urgency.
Serious	Advanced deterioration, overstressing, or breakage may have significantly affected the load-bearing capacity of primary structural components. Local failures are possible and loading restrictions may be necessary. Repairs may need to be carried out on a high-priority basis with urgency.
Critical	Very advanced deterioration, overstressing, or breakage has resulted in localized failure(s) of primary structural components. More widespread failures are possible or likely to occur, and load restrictions should be implemented as necessary. Repairs may need to be carried out on a very high priority basis with strong urgency.

From: *Underwater Investigations, Standard Practice Manual*, ASCE, 2001.

Underwater Concrete Core Testing Report

Memorial Bridge

Concrete Characterization and Condition Assessment – Actual and Future Deterioration

Final Report

Project No. MSL09307

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November 2009

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THE STADIUM® MODEL IS A HELPFUL TOOL TO PREDICT THE FUTURE CONDITIONS OF CONCRETE MATERIALS. HOWEVER, ALL DURABILITY-MODELING PARAMETERS HAVE A STATISTICAL RANGE OF ACCEPTABLE RESULTS. THE MODELING USED IN THIS REPORT USES VALUES AS INPUT PARAMETERS BASED ON TECHNICAL INFORMATION OBTAINED FROM TECHNICAL DATASHEETS. THIS PROVIDES A SINGLE RESULT, WHICH PROVIDES A SIMPLE ANALYSIS EVALUATING CORROSION PROTECTION OPTIONS. PREVIOUS CONDITIONS ARE ASSUMED TO CARRY FORWARD IN THE PREDICTION MODEL; THERE ARE NO ASSURANCES THAT THE STRUCTURE WILL BE EXPOSED TO A SIMILAR ENVIRONMENT AS IN THE PAST.

ALL ANALYSES IN THIS REPORT ARE BASED STRICTLY ON THE CORROSION PROTECTION AND CONDITION OF THE REINFORCED CONCRETE MATERIALS. THE CONDITION APPRAISAL AND ANALYSIS BY NO MEANS CONSTITUTES A STRUCTURAL ENGINEERING CONDITION APPRAISAL OR ANALYSIS. ANY AND ALL RECOMMENDATIONS PRESENTED IN THIS REPORT SHOULD BE VERIFIED AND VALIDATED BY A COMPETENT STRUCTURAL ENGINEER.

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1 Mandate

1.1 Introduction

Materials Service Life, LLC (MSL) was mandated by **Appledore Marine Engineering, Inc.** (AME) to characterize the concrete and to assess the service life of the underwater concrete piers from two bridges located at Portsmouth, New Hampshire. The mandate also covered the assessment of different repair options and recommendations for maintaining the pier in good condition. The service life assessment was done with simulations performed with STADIUM®, a predictive modeling software. The two bridges are:

- Memorial Bridge
- Sarah Mildred Long Bridge

This report presents the concrete characterization and condition assessment for the Memorial Bridge only. A separate report was done for the Sarah Mildred Long Bridge.

The Memorial Bridge is a through truss lift bridge that carries U.S. 1 across the Piscataqua River between Portsmouth, New Hampshire and Badger's Island in Kittery, Maine USA. The bridge was constructed between 1920 and 1923, has a total length of 366.1 m (1,201 ft) and 8.5 m (27.9 ft) width. The main unit is composed by 3 spans, including the lift span between the towers (Figure 1). There are ten approach spans.[1],[2]

Figure 1 – Typical view of the Memorial bridge over the Piscataqua River at Portsmouth, NH[1]

[1] wikipedia
[2] http://nationalbridged.com

1.2 *Objectives and Work Scope*

This investigation was undertaken to identify the cause of the concrete degradation of the underwater concrete piers, to generate information on the residual service-life of the structure and to analyze the influence of different remediation strategies. The work scope included a laboratory investigation and a service life modeling.

The field work, including the cores selection, and the core extraction were performed by Appledore Marine Engineering (AME) personnel. Six 2 ¾-in. diameter cores were extracted in the underwater portion of the piers.

A final report was prepared to summarize the laboratory results, the service life simulations, the conclusions, and the recommendations.

1.2.1 *Laboratory Investigation*

Laboratory testing was conducted on concrete cores received. Results of the laboratory and field investigations were used to predict future performance of the concrete. The laboratory investigation included the following tests:

1. Determination of compressive strength according to ASTM C39 – *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. Compressive strength was determined on two 2 ¾-in. diameter cores taken from two different piers.

2. Determination of ionic diffusion coefficients from the results of transport property tests based on the following procedures:
   - Porosity test according to ASTM C642 – *Standard Test Method for Density, Absorption, and Voids in Hardened Concrete*;
   - Pore solution chemistry analysis;
   - Ion migration test (modified- ASTM C1202 - (05) – *Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*).

   The two companion test samples needed for each test were from two different piers. Moreover, in each selected core, a porosity test sample was selected near an ion migration test sample.

3. Evaluation of the condition of the concrete and possible causes of deterioration. Petrographic examinations were carried out in compliance with guidelines provided in ASTM C856 - *Standard Practice for Petrographic Examination of Hardened Concrete*. Analyses were performed by Niels Thaulow of RJ Lee Group Inc. Petrographic examinations included the determination of carbonation depth using

the phenolphthalein pH-indicator and microscopic observations. The concrete microstructure was observed by optical and scanning electron microscopy (SEM). Petrographic examinations were conducted on one concrete core to obtain information on the concrete and the aggregate properties and to detect any active degradation mechanisms.

4. Determination of total chloride content based on the procedure in ASTM C1152-(04) – *Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete*. Total chloride contents were measured at various depth increments from exposed surface. Chloride ion profiles were determined to assess the severity of chloride ion contamination from exposure to seawater exposure. Two concrete cores were selected, such as they represent different pier and different water level.

5. Determination of the air-void characteristics according to ASTM C457 - *Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*. The test was performed on one core.

1.2.2 *Service Life Modeling*

The service life modeling included three subtasks, described below. Field and laboratory data were used to predict future durability performance and to propose repair options (restoration and maintenance). A state-of-the-art software, called STADIUM®, was used to model future performance against deterioration of concrete. The analysis involved:

1. Determination of past chloride exposure conditions using the chloride ion profiles and concrete transport properties obtained with the laboratory investigation.

2. Prediction of future chloride contamination using STADIUM®. Evaluation of the possible future deterioration based on the data collected during the laboratory testing.

3. Assessment of maintenance and repair options to determine the most effective technical solutions to extend the expected service life of the structure.

2 Laboratory Investigation

Laboratory testing was conducted on the concrete cores received. Tests results are presented in the following sub-sections. The results are presented in this section (section 2) and the discussion on the results is presented in section 3.

2.1 *Visual Inspection of the Cores*

The cores were extracted under the supervision of Appledore Marine Engineering (AME). Six 2 ¾-in. diameter concrete cores from Memorial bridge were received at the laboratory.

From the information received, the cores were taken from different pier and at different elevation. Table 1 presents the list of the cores received along with their location. All concrete cores were identified as written on the concrete core, provided by Appledore Marine Engineering, Inc.

Table 1 - Samples received

Core ID		Pier	Element	EL. (MLLW)	Number of pieces
# 1-1	Memorial Bridge	North	Downstream nose	0′	2
# 1-2	Memorial Bridge	North	Upstream nose	-25′	1
# 1-3	Memorial Bridge	North	Downstream nose	-50′	2
# 2-1	Memorial Bridge	South	Upstream nose	0′	1
# 2-2	Memorial Bridge	South	Downstream nose	-25′	1
# 2-3	Memorial Bridge	South	Upstream nose	-50′	1
# 2-3	Memorial Bridge	South	Upstream nose	-50′	1

Visual observations for all cores are presented in Tables 2. Upon reception, the cores were photo-documented. Pictures of the cores are presented in Appendix A. A complete visual inspection of one core is presented for the samples selected for petrographic examination. These observations are presented in section 2.4.

Table 2 – Visual observations of the cores received from Memorial Bridge

ID	Number of pieces	Diam. (inch)	Length (inch)	General Comment
# 1-1	2	2 ¾	13	- Well consolidated concrete (Dmax = 1 ⅜′′) - Exposed surface was delaminated (apparent broken coarse aggregates) - Cracks parallel to the exposed surface to a depth of 1′′ (cracks propagated through aggregate)
# 1-2	1	2 ¾	12 ⅛	- Well consolidated concrete (Dmax = 2 ⅜′′) - Exposed surface was rough without apparent coarse aggregate
# 1-3	2	2 ¾	13 ¼	- Well consolidated concrete (Dmax = 1 ⅝′′) - Exposed surface was covered by plywood - White deposits at 7 ½′′ depth
# 2-1	1	2 ¾	8	- Well consolidated concrete (Dmax = 2′′) - Exposed surface was rough without apparent coarse aggregate - Crack perpendicular to the exposed surface, 1′′ long from surface, at cement-paste / aggregate interface
# 2-2	1	2 ¾	12 ¼	- Well consolidated concrete (Dmax = 2 ⅜′′) - Exposed surface was rough without apparent coarse aggregate - Crack parallel to the exposed surface at ¼′′ depth (white deposit within the crack)
# 2-3	1	2 ¾	12 ¼	- Well consolidated concrete (Dmax = 2′′) - Exposed surface was rough without apparent coarse aggregate

2.2 *Compressive strength*

Compressive strength determination was performed in accordance with ASTM C39 – *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. Individual results are summarized in Table 3, expressed in psi and MPa. From Table 3, the average compressive strength is 3,270 psi. However, variation is observed from the two results.

Table 3 – Compressive strength results

Core I.D.	Bridge	Sample position in core⁺, inch	Compressive Strength, psi (MPa)†
# 1-2	Memorial Bridge	5 ½′′ to 10 ⅝′′	4,785 (33.0)
# 2-1	Memorial Bridge	¾′′ to 5 ⅞′′	1,755 (12.1)

Note ⁺: depth measured from the exposed surface of core
Note *: as received conditions
Note †: 1 MPa = 145 psi

2.3 *Air-void characteristics*

The air-void characteristics were determined according to ASTM C457 - *Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*. The characteristics measured with this test allows evaluating the concrete's ability to adequately resist frost action such as freezing and thawing cycles in saturated conditions in the presence or absence of deicing salts.

Test was performed on one core by bridge. The results of the tests are presented in Table 4. In Table 4, the measured air content and spacing factor are below 2.5% and over 0.020 inch (500 μm), respectively.

Table 4 - Parameters of the Air-Void System in Hardened Concrete

Core I.D.	Bridge	Paste Content, %	Air Content, %	Specific Surface, inch^{-1} (mm^{-1})	Spacing Factor, inch (μm)
# 1-2	Memorial Bridge	24.2	2.4	323 (12.7)	0.020 (506)

2.4 *Transport properties*

To generate information on the transport properties of concrete, the ionic diffusion coefficients were determined with the porosity test and the ion migration test. These tests were performed on concrete specimens cut from the extracted cores. The two companion test samples needed for each test were from two different piers.

2.4.1 *Porosity*

Porosity measurements were performed according to ASTM C642 – (06) – *Standard Test Method for Density, Absorption, and Voids in Hardened Concrete*. Porosity corresponds to the total volume of voids that can be saturated with water. In addition to provide information on the quality of in-place concrete, porosity values were used as input parameters in STADIUM®-IDC to determine diffusion coefficients and in STADIUM® to simulate future contaminant ingress. The selected cores and their respective results are summarized in Table 5.

As can be seen in Table 5, porosity results are variable, from 9.1% to 13.0%. The absorption results are also variable, from 3.8% to 5.7%, which correlates with the porosity results. Based on these results, the concrete seems to be of good quality.

Table 5 – Absorption and Porosity Results

Core I.D.	Bridge	Sample position in core⁺, inch	Absorption, %	Porosity, %
# 1-3	Memorial Bridge	1 ⅛′′ to 5 ⅛′′	5.7	13.0
# 2-3	Memorial Bridge	2 ½′′ to 6 ¾′′	3.8	9.1

Note ⁺: depth measured from the exposed surface of core

2.4.2 *Ionic Diffusion Coefficients*

Ionic migration tests were performed to characterize the ionic diffusion properties of the concrete cores extracted from the piers. The test used was a modified (and improved) version of ASTM C1202 - (05) – *Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*.

Ion transport through a saturated concrete specimen was accelerated by applying an electrical potential to the test cell. During testing, which lasts usually 14 days, the current passing through the sample was measured. Migration test results, as well as porosity, were used to determine the diffusion coefficients using STADIUM®-IDC, a specialized version of STADIUM®. The length of the samples was extended to 3′′ rather than the

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usual 2'' length in order to avoid direct path along the cement paste-aggregate interface. The selected cores and their respective results are presented in Table 6. The results vary from 9.5 to 19.0 x 10^{-11} m^2/s, for an average of 14 x 10^{-11} m^2/s.

Table 6– OH^- Diffusion Coefficient Results

Core I.D.	Bridge	Sample position in core⁺, inch	OH^- Diffusion coefficient, x 10^{-11} m^2/s	Average, x 10^{-11} m^2/s
# 1-3	Memorial Bridge	8 ¼'' to 11 ¼''	19.0	14
# 2-3	Memorial Bridge	6 ¾'' to 9 ⅝''	9.5	

Note $^+$: depth measured from the exposed surface of core

2.5 *Petrographic examinations*

One core was selected for petrographic examination. Sample was cross-sectioned and one surface was coated with the pH indicator phenolphthalein. Two thin sections were then prepared from the core, one from the exposed surface and one from the interior (a total of 2 examinations was performed). Niels Thaulow at RJ Lee Group performed these examinations.

The thin sections were analyzed in accordance with ASTM C 856-04 *Standard Practice for Petrographic Examination of Hardened Concrete* using visual examinations, an optical stereo-microscope, a polarized light microscope (PLM) and a scanning electron microscope (SEM) with energy dispersive spectrometry (EDS). The water/cement ratio (W/C) was determined by fluorescence microscopy and the composition of the concrete was calculated based on W/C and estimation of the cement paste content.

2.5.1 *Visual Observations of Samples*

The visual observations are a compilation of observations made visually on the core in as-received condition and with an optical stereo-microscope on the cut face of the core.

North Pier of Memorial Bridge, core #1-1 (RJLG ID 3638015)

The received half-core was 2 ¾" in diameter and approximately 12 ¾" in length. It was broken into two pieces at approximately 9 ½" from the exposed surface. The top exposed surface had marine growth, exposed aggregate, and small cracks. The bottom was a fractured surface. No reinforced steel was present. The paste was light gray in color and even in appearance. The concrete was well consolidated. The coarse aggregate was mostly rounded. The maximum aggregate size was about 1 ½". The exposed surface was carbonated to a depth of 1 mm, but patch carbonation, as indicated by the dark pink color, was seen at approximately 2 ½" deep. Section location and the phenolphthalein test are shown in Figure B1 and Figure B2 in Appendix B.

2.5.2 *Microscopy Observations*

Microscopy observations were performed on thin sections extracted from selected areas.

North Pier of Memorial Bridge, 1-1, RJLG ID 3638015

Thin sections were prepared from the top exposed surface (noted as "T") and from the interior of the core (noted as "B") perpendicular to the exposed surface. The concrete was not air-entrained and was dense with a water/cement ratio of approximately 0.50. The cement was a Portland cement and fully hydrated. The cement paste content was estimated as 30% by volume. No fly ash or slag was seen. The coarse aggregate consisted of predominately feldspar and quartz. The fine aggregate was quartz. No alkali silica reaction was seen. The exposed surface had severe external sulfate attack seen as cracking parallel to the exposed surface and ettringite in cracks and voids. Monosulfate was seen in the interior sample. Calcium carbonate was seen in cracks near the exposed surface and calcium hydroxide was seen in the non-carbonated paste throughout the core. Friedel's salt, a sign of chloride ingress, was seen in the top section of the core. In Appendix B, images from the petrographic microscope are shown in Figure B3 through Figure B8 and images from the SEM are shown in Figure B9 through Figure B37.

2.5.3 *Concrete composition*

The composition of the concrete was assessed during the petrographic examinations and is summarized in Table 7. The cement content is calculated based on the estimated paste volume and water-cement ratio assuming a specific gravity of 3.15 for the cement. The concrete has a water/cement ratio of 0.50, which correlates with the porosity results but not the ionic diffusion coefficient results. The concrete analyzed is of fair quality based on all results.

Table 7 – Composition of the Concrete

Core I.D.	Bridge	Observed properties		Calculated Cement Content*, lb/yd³ (kg/m³)
		Water/Cement ratio	Paste Content, %	
# 1-1	Memorial Bridge	0.50	30	619 (367)

Note *: The cement content is calculated based on the estimated paste volume and w/c ratio assuming a specific gravity of 3.15 for the cement.

2.6 *Total Chloride Content Profiles – Test Results*

Total chloride ion content was measured, at various depth increments, based on the procedure described in ASTM C1152 – *Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete*. The concrete was ground at different depth to produce a fine powder prior to chloride determination. For each chloride content determination, a minimum of 20 grams of concrete was ground. Chloride ions were digested using a diluted nitric acid. Diluted nitric acid was prepared with a commercial laboratory concentrated nitric acid (normal concentration of 65%). The mix was 1 volume of 65% HNO_3 to 9 volumes of distilled water. After the oven-drying period, 10 grams of dried powder sample was weighed and placed into a 150-ml beaker. The 100 ml diluted nitric acid, heated at 176°F (80°C), was poured into each beaker and mixed with the powder sample. One hour after adding acid, the dissolved solution was then vacuum filtered using filter paper (47 mm Ø55, Cat. no. 1001 055). The filtrate was collected in plastic sample bottle for chloride concentration analysis. Chloride content was determined using an automatic titrator (Mettler DL21).

Chloride profiles were performed on two concrete cores selected from the received cores (see Table 1). The selected concrete cores represent different piers and different water level. Figure 2 presents the chloride profiles determined on the selected concrete cores. On the graphs, 0 on the X-axis represents the surface exposed to seawater. In this figure, it can be observed that, near the surface, the chloride concentrations are relatively high (around 7,000 ppm).

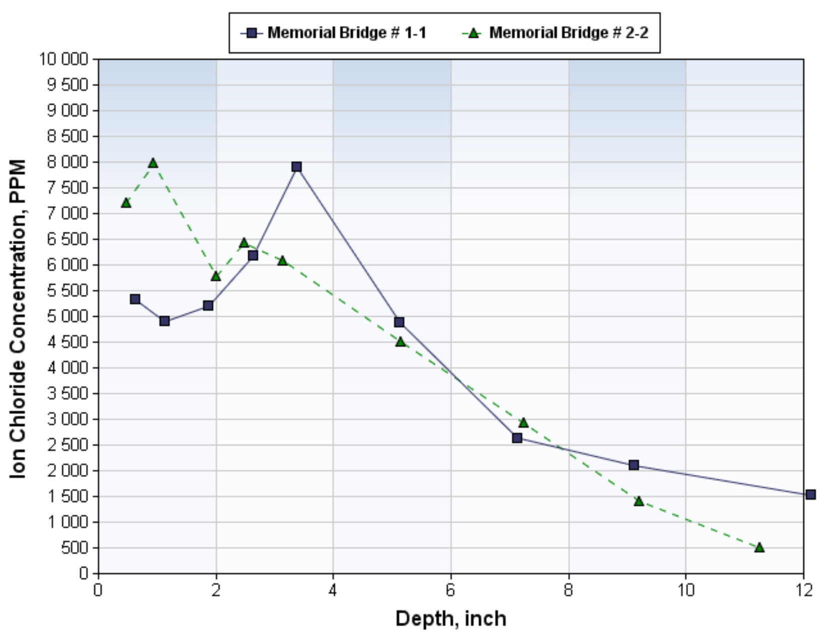

Figure 2 – Experimental chloride profiles – Memorial Bridge after 89 years of exposure

3 Discussions – Results and Observations

Different tests were performed to assess the characteristics of the concrete, the contamination level and possible degradation mechanisms inside the concrete extracted from the Memorial Bridge over the Piscataqua River at Portsmouth, NH.

3.1 Compressive strength

The compressive strength results are variable from one location to the other (1,755 and 4,785 psi (12.1 and 33.0 MPa)). The test on core #2-1 failed from the capped side which was closed to the exposed surface. Visual observations showed damage at the exposed surface that can explain the unsuitable failure and the lower results for this sample, i.e. 1,755 psi (12.1 MPa). The unit weight of the tested sample also indicates a lower concrete quality for the sample #2-1 compared with sample #1-2[3]. It is thus considered that the result from core #1-2, i.e. 4,785 psi (33.0 MPa), is more representative of the in-place undamaged mass concrete of the piers of Memorial Bridge.

3.2 Air void characteristics

These results suggest that pier's concrete was not properly air entrained. The air-void network does not possess the required characteristics to protect the concrete of the piers against the effects of freezing and thawing in saturated conditions or against deicer salt scaling. Suggested values of 0.008 inch (200 µm) for the spacing factor and 6% for the air content are given in the document ACI 201.2R *Guide to Durable Concrete* to protect concrete against frost action.

3.3 Transport properties

The concrete from both piers have porosity around 13%. The porosity results are indicative of good quality concrete. Good quality normal-weight laboratory-produced concrete with a 0.45 water-binder ratio generally shows 11% to 12% porosity. It should be noted that the nominal diameter of the coarse aggregates is large (i.e. about 2′′ in South Pier of Memorial Bridge, see Table 2). Thus the test could not be representative of the in-place concrete since the cores had a diameter of 2 ¾′′, in particular for the South Pier of Memorial Bridge that exhibits higher nominal diameter for the coarse aggregate (Table 2). The lower porosity of core #2-3 can be explained by the presence of large quantity of aggregates, which have a lower porosity than the mortar matrix. Since aggregates are not porous, more aggregates in the sample usually lead to lower porosity.

[3] Sample #2-1 unit weight: 139 lb/ft3; Sample #1-2 unit weight: 156 lb/ft3;

The ionic diffusion coefficient varies from one location to the other. The variability in the ionic diffusion coefficient results is consistent with the variability found for the porosity results given in Table 5 (both samples were extracted from the same core). A variability is observed in this concrete as it was for the porosity and the compressive strength.

The ionic diffusion coefficients indicate that the concrete is more porous than a typical 0.45 water-cement ratio concrete. Based on this affirmation, the ionic diffusion coefficient does not correlate the quality of concrete with the porosity. From the ionic diffusion coefficient results, the concrete from these structures could be considered porous, since good quality laboratory-produced concrete having a water/binder ratio of 0.45 normally shows values from 10 to 13 x 10^{-11} m^2/s.

3.4 Petrographic examination

The North Pier of Memorial Bridge showed signs of severe external sulfate attack to a depth of 1 ½" at the waterline. The potential for more deterioration in the future is still possible.

At the exposed surface, there were cracks parallel to the surface and a portion of the cement paste was converted to magnesium silicate, which has no strength (see Figure 3). Calcium carbonate, magnesium hydroxide (see Figure 4), and ettringite (see Figure 5) were found in the cracks near the exposed surface. Ettringite is a sign of external sulfate attack, whereas the calcium carbonate and magnesium hydroxide result from exposure to seawater.

There were no signs of alkali silica reaction. The interior of the pier was sound concrete with fully-hydrated Portland cement.

There was evidence of chloride ingress at the exposed surface in the form of Friedel's salt and chloride in the paste. Chloride ingress can cause corrosion of the steel reinforcements without deleterious effects in the paste. The concrete was not air-entrained, which may exacerbate cracking at the waterline and above due to freezing and thawing.

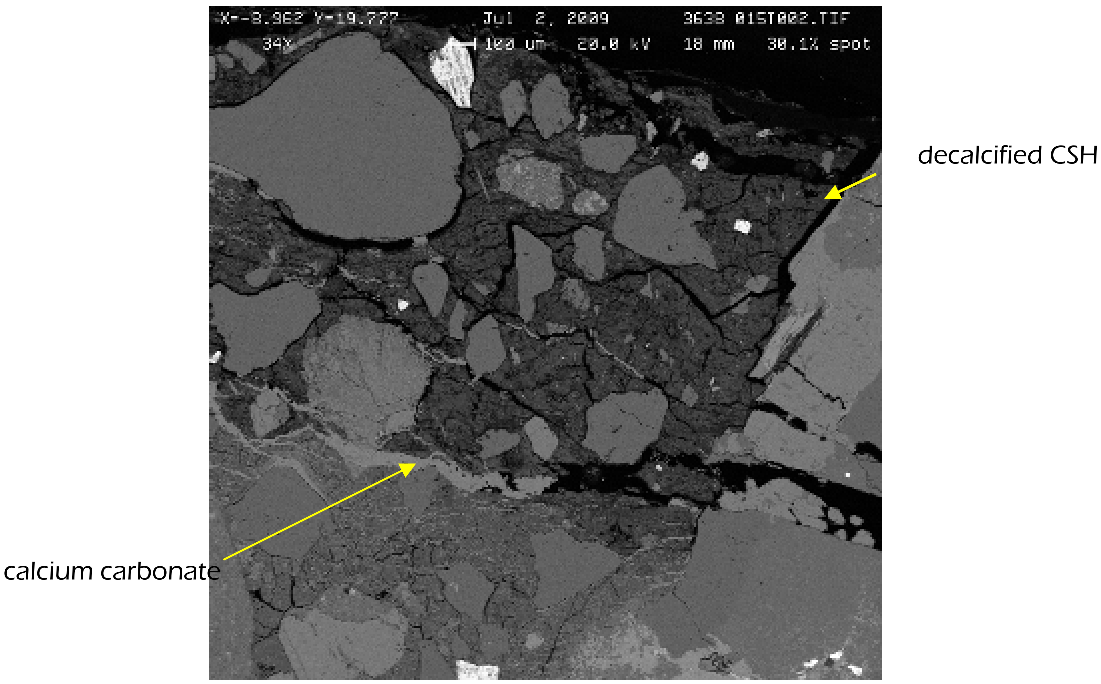

Figure 3 - North Pier of Memorial Bridge, core #1-1 (RJLG ID 3638015T). SEM image showing many cracks parallel to the exposed surface due to external sulfate attack. Near the surface, decalcified CSH has been converted to magnesium silicate with aluminum, which has no strength. Calcium carbonate is seen in the crack. The area below the crack is rich in magnesium.

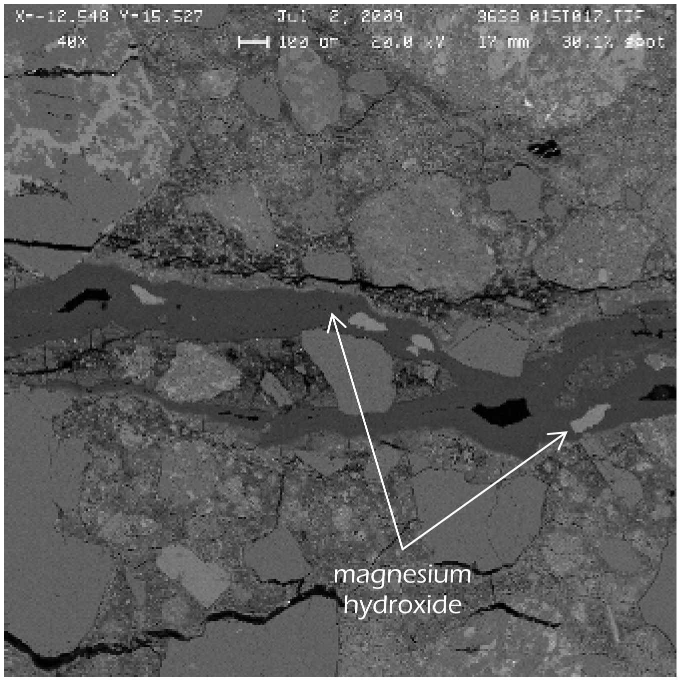

Figure 4 - North Pier of Memorial Bridge, core #1-1 (RJLG ID 3638015T). SEM image showing crystalline magnesium hydroxide (Brucite) in a crack.

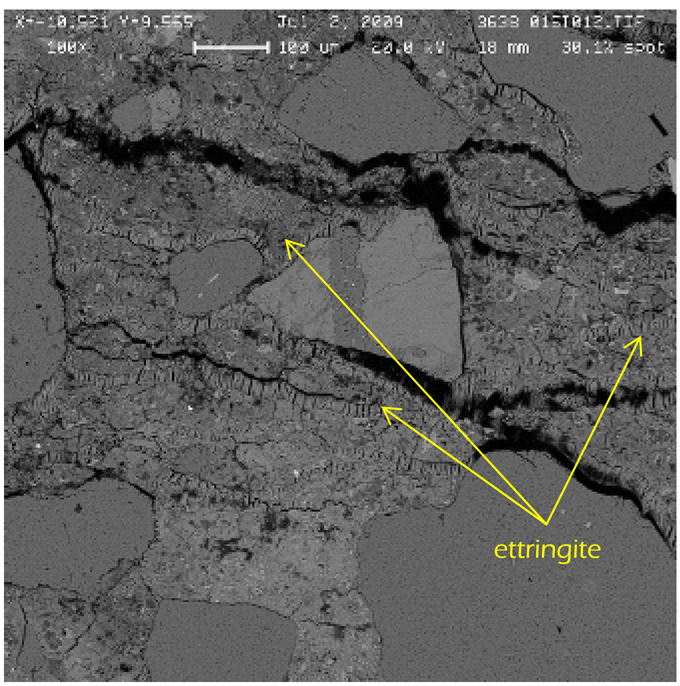

Figure 5 - North Pier of Memorial Bridge, core #1-1 (RJLG ID 3638015T). SEM image showing ettringite filling multiple cracks.

3.5 Chloride profiles

Chloride profiles have shown in Figure 2 (see section 2.6) show that the investigated piers are contaminated with chlorides. From the results, the concrete is contaminated with chlorides to a depth of at least 12 inches. Chloride profiles in Figure 2 suggest that the concrete from piers is exposed to high salinity water.

4 Corrosion initiation

The critical chloride concentration required to initiate the corrosion reaction has been extensively investigated over the past decades. Moreover, the critical chloride threshold for corrosion initiation is not a fixed value. Recent investigations suggest that the threshold chloride concentration is also influenced by the quality of the steel (plain steel, epoxy-coated, or stainless).[4] Furthermore, for a given type of steel, threshold values tend to vary quite significantly.[5,6] For instance, in their comprehensive review, Alonso et al.[2] found that values reported for black steel ranged from 0.25% to 3.0% of chloride per mass of binder. A total chloride content above 0.30% of the cement mass is often considered sufficient to initiate corrosion of black steel[7] in concrete that is initially chloride-free. This threshold is used by the Federal Highway Administration (FHWA) but is, based on the literature data, a conservative assumption since it is in the lower part of the bracket.

Tests performed by MSL have yielded a threshold value within this range. The corrosion initiation threshold established by MSL is 0.50 ± 0.05% of the cement mass[8]. This critical chloride content will be used in the following analysis.

Using a critical chloride content threshold of 0.50 ± 0.05% per mass of cement, for a concrete having a cement content of 619 lb/yd^3 (367 kg/m^3), based on the petrographic examinations, and a bulk dry specific gravity of 2.30 (calculated from the porosity results), the chloride threshold would be 0.072 % by mass of dry concrete (720 ppm).

The ion chloride profiles in Figure 2 indicate that the chloride content is over the critical value of 720 ppm up to 12 in. depth. However, the corrosion initiated by chloride contamination should propagate slowly where concrete element is totally immerged. The fully saturation of concrete leads to the lack of oxygen around the embedded rebars. The

[4] Federal Highway Administration (FHWA), *Corrosion Evaluation of Epoxy-coated, Metallic-clad and Solid Metallic Reinforcing Bars in Concrete*, Report No. FHWA RD 98 153. December 1998.
[5] Alonso, C. et al. 2000, 'Chloride Threshold Values to Depassivate Reinforcing Bars Embedded in a Standardized OPC Mortar', *Cement and Concrete Research*, Vol. 30, pp. 1047-1055.
[6] Bentur, A., Diamond, S., Berke, N. (1999) *Steel Corrosion in Concrete - Fundamentals and Civil Engineering Practice*, E & FN Spon.
[7] Rosenberg, A. et al. 1989, 'Mechanisms of Corrosion of Steel in Concrete', *Materials Science of Concrete*, Vol. 1, J. Skalny ed., American Ceramic Society, Westerville (USA), pp. 285-313.
[8] Henocq et al., 2007, '*Determination of the Chloride Content Threshold to Initiate Steel Corrosion*', Proceedings of the 5th International Essen Workshop – Transport in Concrete:Nano to Macrostructure, Max J. Setzer Editor, June 2007.

reduction reactions are slowed, if not stopped, without supply of oxygen at the rebar. Under these conditions, the reduction reaction controls the corrosion rate which is maintained low. However, in tidal zone, oxygen is available and corrosion can propagate faster.

5 Service Life Modeling - Time to Corrosion Initiation

5.1 Exposure conditions

Numerical simulations were run to predict future chloride ingress and concrete degradations. For the simulations, it was considered that the piers were massive and that at 30 inches, the concrete condition was stable. Thus only the first 27 ½ inches, from the exposed surface, were simulated. The parameters for the modeling have been presented in section 2.

The temperature of the sea water was considered since the cores were below the Mean Lower Low Water elevation (Table 1). Temperature was fixed based on data found for Wells, ME, located at 20 miles from Portsmouth, NH (source: http://tidesand currents.noaa.gov). Numerical simulations were run assuming the concrete was immersed in seawater with a salinity of 34‰. The concrete was exposed to 100% relative humidity from the exposed surface. The average temperature used in the simulations is 49 ± 12.5°F. Figure 6 presents the cyclic temperature used in the numerical simulations.

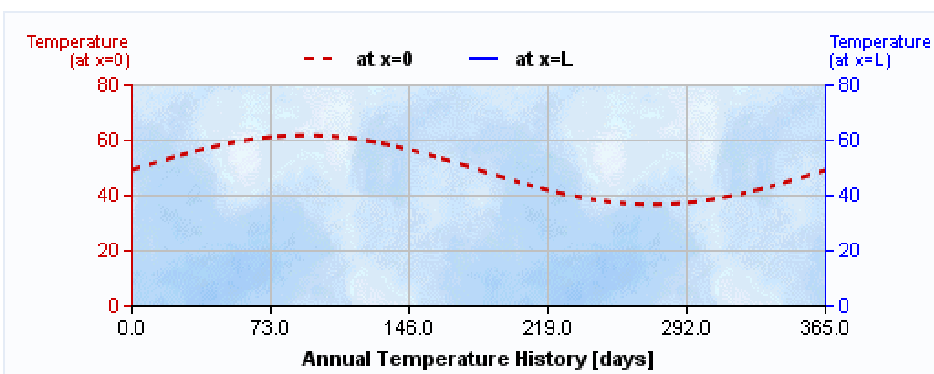

Figure 6 - Cyclic temperature used in the simulation

Simulations were run for 89 years. From the salinity, the seawater had 530.7 mmol/L of chloride, 27.2 mmol/L of sulfate, and 51.6 mmol/L of magnesium. The concentration of chloride is considered high while for the two other ion species, the concentrations are considered low. This is typical for seawater composition on the east coast.

The mix used in the simulations was based on the results of the petrographic examinations that indicated that the water-binder ratio was approximately 0.50 and the paste content was around 30%. A 0.45 water-binder mix without supplementary cementitious materials was selected. Due to the high chloride binding potential of the concrete (see Figure 2), the cement used in the simulation was rich in SO_3 and Al_2O_3.

5.2 *Numerical Model Validation*

Exposure conditions were determined for both investigated structures. Figure 7 presents the experimental chloride profiles along with the numerical simulation results used to reproduce them using the exposure conditions presented earlier. A red line represents numerical simulations. It can be seen that STADIUM® simulations reproduced the actual chloride penetration relatively well. However, the average ionic diffusion coefficient had to be increased to a value of 30 x 10^{-11} m^2/s compared to a maximum value measured of 19 x 10^{-11} m^2/s. This is explained by the variability in the results and possible damage in concrete where the chloride profiles were determined. This would result in a higher ionic diffusion coefficient than the measured one where it was selected deep enough inside the structure where the concrete was sound.

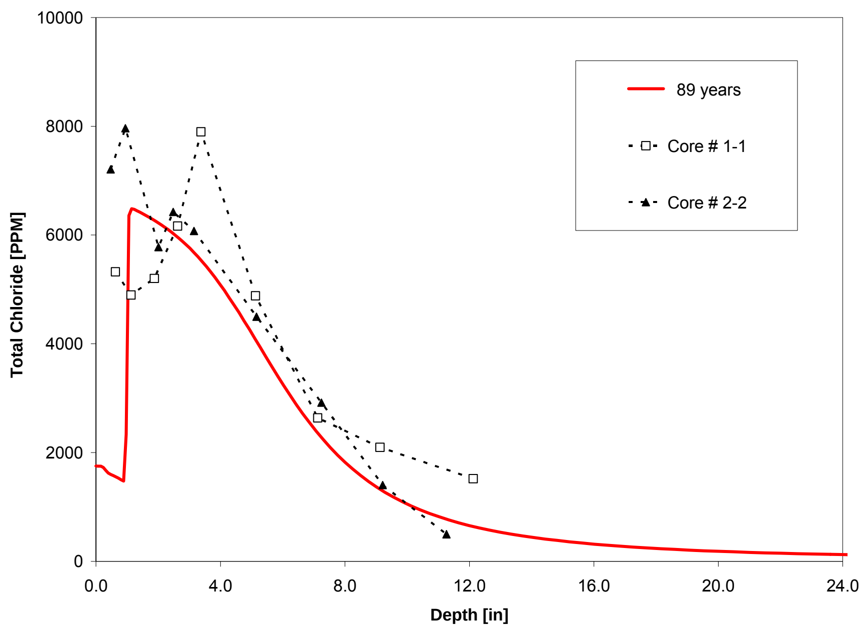

Figure 7 - Chloride profiles measured on Memorial Bridge and STADIUM® numerical simulation

5.3 *Future chloride penetration – no repairs*

Since chloride have already reached the rebar position (2 ½ in. for Memorial Bridge), future chloride ingress for potential corrosion initiation is irrelevant. Nevertheless, simulations were performed for 50 years in the future and for different depth in concrete, the time for the chloride concentration to reach critical level is presented.

From Table 8, and considering the rebar concrete covers, the corrosion should have initiated early in the age of the structure. The corrosion propagation could have been slowed down quite considerably since the concrete is in the immersed part of the piers. In the immersed concrete, the oxygen, required for the corrosion reaction, availability is low and thus the corrosion reactions are hindered.

Table 8 - Time to reach critical chloride concentration for corrosion initiation

Structure	Concrete Depth						
	2.5 in.	4 in.	6 in.	8 in.	10 in.	12 in.	18 in.
	Years from the time of construction*						
Memorial Bridge	4	11	26	47	74	107	>139

Note *: present concrete age is 89 years for Memorial Bridge.

It should be noted that the critical chloride level will be reached at a depth of 12 in. in 18 years from 2009.

5.4 *Chemical degradation – no repairs*

The analysis of chemical degradation was carried out for the concrete of piers. A numerical simulation was performed to provide information on the microstructural alterations of concrete resulting from the exposure to seawater. The sulfate and magnesium ions, present in seawater, are likely to react with the hydrated cement paste to form new compounds that have detrimental effect on the microstructure of concrete.

Upon exposure to seawater, a number of hydrated and unhydrated phases in the concrete tend to react with the migrating sulfate ions. These phases are *calcium hydroxide, tricalcium aluminate, and monosulfoaluminate*. The chemical reactions between these phases and the sulfate ions result in the formation of new products that have little cementing properties. As pointed out by Taylor[9], the formation or the dissolution of phases during sulfate attack can lead to strength loss, expansion cracking, and, ultimately, disintegration of the solid.

[9] Taylor H.F.W., (1990), *Cement Chemistry*, Academic Press Inc., San Diego, 475 p.

Portlandite is the most soluble phase and acts as a buffer for the pore solution of concrete. The dissolution of portlandite supplies OH^- and Ca^{2+} ions and helps to maintain the pH and equilibrium of the solution. When the portlandite is dissolved, the calcium of the CSH phase starts to dissolve and the paste is weakened. The gradual dissolution increases the porosity of the paste and decreases its resistance, which causes gradual erosion of the concrete surface. Hydroxyl ions (OH^-) can be leached out by soft water as the concentrations tend to equilibrate. In seawater, the dissolution occurs even faster, because the diffusion of chlorides and sulfates (negatively charged ions) is accompanied by a counter diffusion of hydroxyl ions.

External sulfate ions can react with the aluminum phases of the hydrated cement paste to form ettringite. The reaction is essentially driven by a dissolution/precipitation process. The formation of ettringite from tricalcium aluminate and calcium hydroxide leads to a volume increase of the solid about 280% (Clifton and Pommersheim[10]).

The environmental sulfate ions may also react with the crystalline calcium hydroxide present in the hydrated cement to yield crystalline gypsum and brucite (the formation of brucite occurs only if concrete is exposed to magnesium and sulfate-bearing solution). This two-step chemical reaction first involves the dissolution of portlandite and then the formation of gypsum (and/or brucite). The formation of gypsum from calcium hydroxide yields to a volume increase of the solid about 120% (Clifton and Pommersheim[7]). The formation of brucite is usually not expansive.

The state of solid phases presented earlier (Portlandite, Ettringite and Brucite) was assessed using STADIUM® modeling software. These results are best shown in graphical representation. At an age of 89 years, the solid phases for Memorial Bridge are given in Figure 8 while in 50 years in the future they are shown in Figure 9. Currently, based on STADIUM® modeling, the formation of ettringite has reached 1 ⅛ in. This depth corresponds to the dissolution of portlandite. For this depth, there is a risk of concrete cracking. The formation of brucite, another expensive product, has reached ⅜ in.

In the next 50 years, the chemical degradation will be limited to the first 1 ½ in. No major chemical degradation should occur deep inside the concrete. In 50 years from 2009, the ettringite, and the dissolution of portlandite, will have reached 1 ¼ in. In 50 years, brucite should have reached a depth of ½ in.

[10] Clifton J.R., Pommersheim J.M., (1994), *Sulfate Attack of Cementitious Materials: Volumetric Relations and Expansions*, National Institute of Standards and Technology, NISTIR 5390, 20 p.

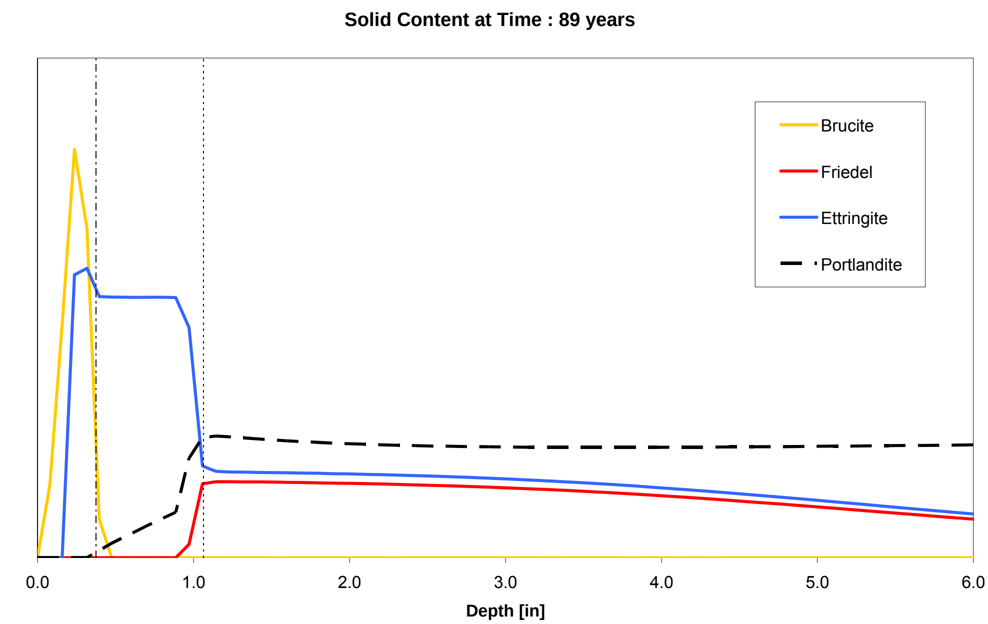

Figure 8 - Solid Phases – Memorial Bridge – Today

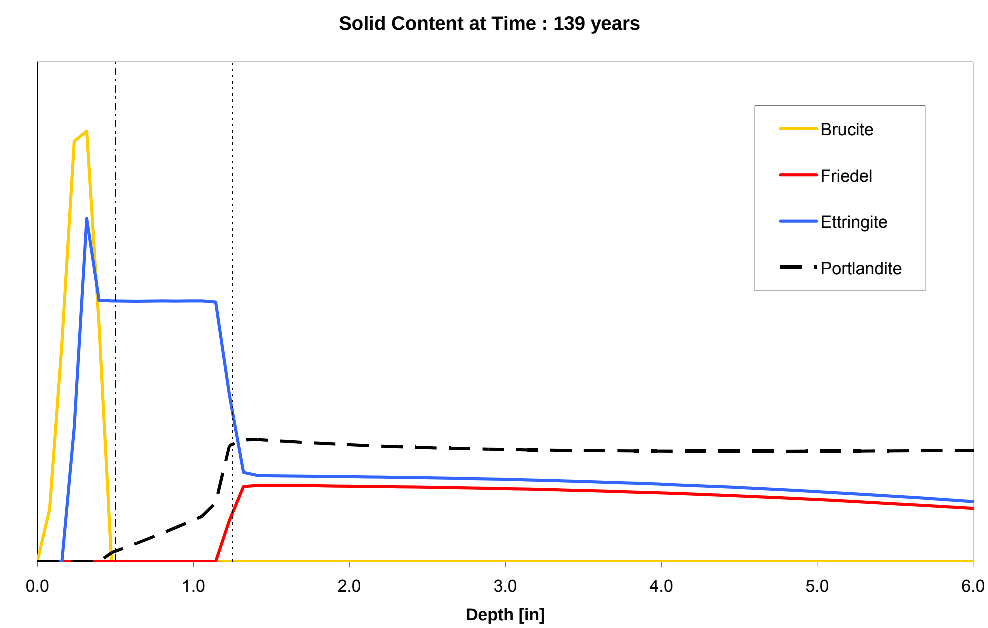

Figure 9 - Solid Phases – Memorial Bridge – 50 years in the future

5.5 Repair Options

Two different repair options were considered and their relative durability regarding the risk for corrosion initiation was assessed. The information provided could be useful in evaluating the required depth of chloride removal to slow, or even stop, the penetration of chloride deeper in the old concrete. It is also considered that near the lower tidal zone, the concrete is as contaminated as the immersed concrete. Thus in that area, the piers require repairs and the selection of the repair materials and the depth of repair is important.

In the simulations, it was considered that new rebar were placed with a concrete cover of 3 in. Two different repair materials were considered:

1. Plain Type 1 cement concrete with a water/binder ratio of 0.35
2. Type 1 cement concrete with 20% fly ash and a water/binder ratio of 0.45

The first mixture is more a typical low water/binder ratio repair materials while the second mixture is believed to be better repair materials for the structures. In fact, mixture 2 should have mechanical properties closer to the structures' concrete, which would allow a better compatibility between the two concrete (old and new). In both cases, the repair concrete should be properly air entrained to resist the effects of freezing and thawing in saturated conditions with or without deicer salt.

5.5.1 *6-inch repair*

Figure 10 presents the chloride content evolution with time for different depths inside the concrete for a 6-inch repair with mixture 2. This type of repair stops the increase of chloride content to a depth of 10 in. or more. However, since the concrete is highly contaminated, the time to initiate corrosion for a concrete cover of 3 in. is not prevented for more than 28 years.

Moreover, the results for Mixture 1 are not presented since the difference between both concrete in preventing corrosion is negligible. Still, it is a more appropriate choice to repair with mixture 2 since its mechanical properties should be similar to the in-place concrete.

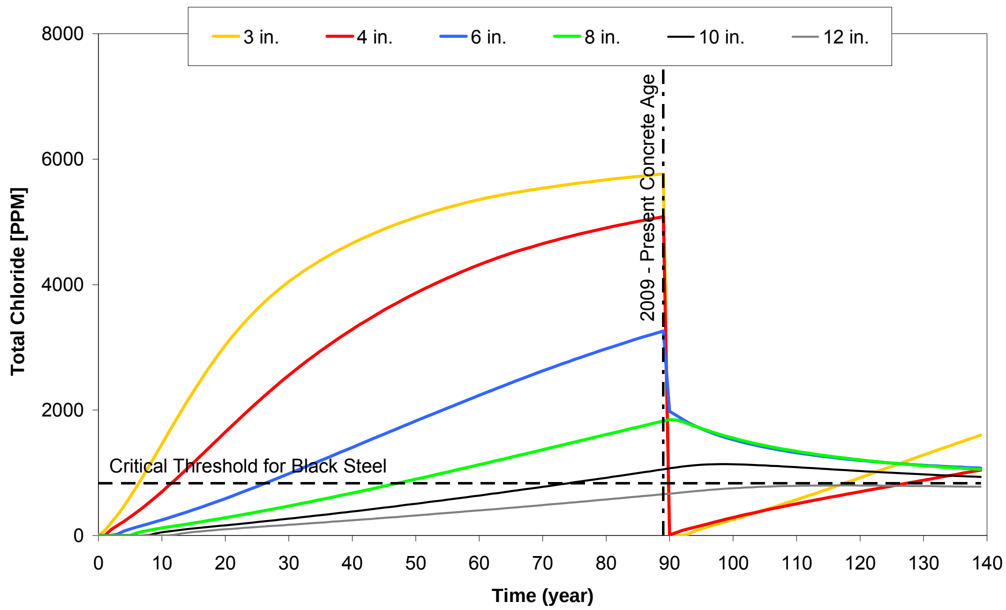

Figure 10 - Memorial Bridge – 6-inch repair – Mixture 2

5.5.2 *10-inch repair*

Figure 11 presents the chloride content evolution with time for different depth inside the concrete for each structure for a 10-inch repair with mixture 2. Analysis of the figure clearly shows the beneficial influence of removing old contaminated-concrete to higher depth. This type of repair stops the chloride ingress for a depth of 10 in. or more like a 6-inch repair. However, the impact of this deeper concrete repair is greater.

In the figure, it is shown that the chloride concentration at a depth of 3 in. inside the repair materials will reach the critical concentration in less than 50 years (i.e., 36 years). Thus, the simulations indicate that the repair material should have a certain quantity of corrosion inhibitor to increase the critical chloride content for corrosion. From our calculation, the addition of 1 gal/yd^3 of corrosion inhibitor should be sufficient to provide 50 years of corrosion protection. At 4 in. inside the repair materials, the critical chloride concentration should be reached after more than 50 years.

Moreover, the comments mentioned in the 6-inch repair regarding the mixture choice still apply.

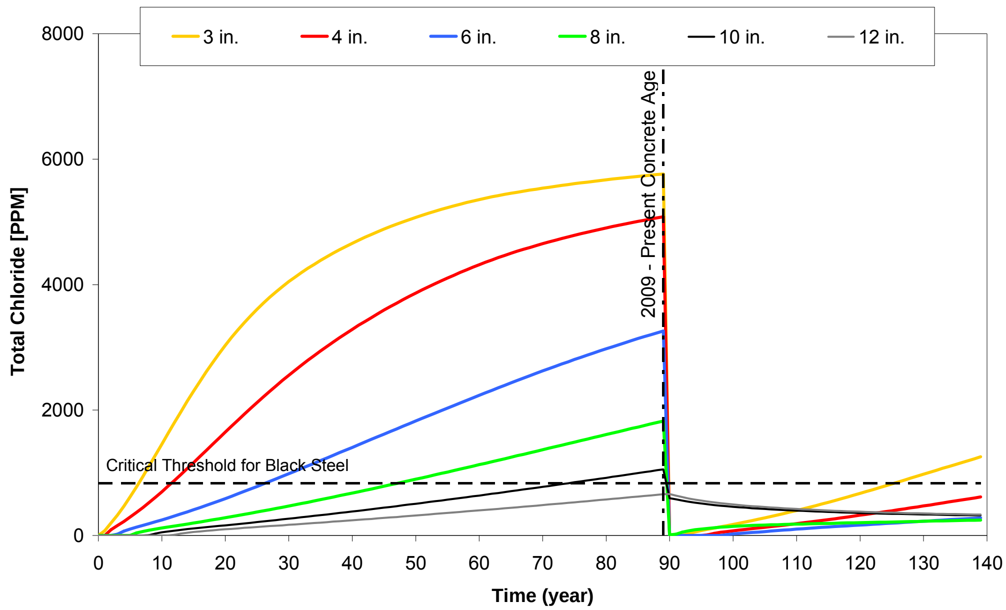

Figure 11 - Memorial Bridge – 10-inch repair – Mixture 2

6 Conclusion

The Memorial Bridge is located in Portsmouth, NH. The piers' concrete was investigated and the service life was assessed. For this mandate, a total of 6 cores from the Memorial Bridge were extracted by Appledore Marine Engineering and sent to our laboratory for concrete test characterization. All concrete were taken on or below the low water level.

The concrete characterization revealed that the concrete inside the piers is sound and that the compressive strength of 4,785 psi (33.0 MPa) is more representative of the in-place mass concrete of the piers. Near the surface, the concrete is showing lower compressive strength. Moreover, the air void network characteristics revealed that the concrete does not have the properties to be frost resistant, thus making it susceptible for frost damage.

The transport properties (i.e., porosity and ionic diffusion coefficient) show that the concrete has variable properties. However, the porosity indicates that the concrete is of good quality while the ionic diffusion coefficients suggest that the concrete is porous. The chloride profiles show that the concrete is contaminated by external chlorides and is exposed to high salinity seawater. The concrete from both structures is contaminated to a depth of approximately 12 inches. Near the rebar position, the chloride concentration is high enough to initiate and promote corrosion. However, the corrosion initiated by chloride contamination should have propagated slowly where concrete element is totally immerged due to lower oxygen concentration. Thus, the risk for corrosion underwater is low. This is observed in concrete located below the water level for a similar structure studied in same mandate[11]. At these depths, the oxygen availability is scarce.

Petrographic examinations were performed to characterize the concrete and to provide a general assessment of the current condition of the concrete, below the low water level, and the potential mechanisms of degradation. For the North Pier of the Memorial Bridge, the concrete showed signs of severe external sulfate attack to a depth of 1½" at the waterline. There were no signs of alkali silica reaction and delayed ettringite formation (DEF). The concrete in the interior of the piers was sound. The petrographic examination showed signs of chloride contaminations. It was also observed that the concrete was not air-entrained. Finally, the concrete had a water-binder ratio of 0.50.

Different numerical simulations were performed to assess the concrete state in the future (50 years in the future) and to evaluate different repair depths. Based on the contamination results, the concrete is exposed to high salinity seawater (i.e., high chloride concentration and low concentrations of sulfate and magnesium). Modeling was thus performed to predict how the degradation will progress inside the concrete within the next 50 years. In addition, numerical modeling was performed to help in the elaboration of repair strategies.

Based on the numerical simulations, the chemical degradations (mainly sulfate attack) will not affect concrete deeper than 2 inches from the exposed surface for the next 50 years. From the petrographic examinations, there are no signs of alkali silica reaction or DEF inside the concrete and there is no indication that these degradation mechanisms will occur in the next 50 years. Moreover, the concrete below the low water level is not submitted to freeze-thaw cycles and the corrosion rate seems low. Thus, the remaining service life of this part of the piers if no repair is done should be higher than 50 years, assuming the structure can tolerate 2-inches of concrete loss, the corrosion rate remain low and the exposure conditions do not change.

Repair options were evaluated for concrete area where the corrosion can propagate at high rate due to the oxygen availability or where concrete can be submitted to the effect of freeze-thaw cycles. Two different depths of the concrete repair were considered to evaluate the impact of the remaining chlorides in old concrete. The simulations considered those two repair materials:

1. Plain Type 1 cement concrete with a water/binder ratio of 0.35 and air entrained;
2. Type 1 cement concrete with 20% fly ash, a water/binder ratio of 0.45 and air entrained.

---

[11] Materials Service Life, LLC, Memorial Bridge and Sarah Mildred Long Bridge - Concrete Characterization and Condition Assessment - Actual and Future Deterioration, Project No.MSL09307, August 2009.

As explained in the report, the second mixture is believed to be a better repair material for this structure for many reasons (i.e., this mixture should have mechanical properties closer to the structure's concretes).

The different repair scenarios were simulated. The result revealed that a 6-inch repair stops the increase of chloride content to a depth of 10 in. or more. However, since the structure's concretes are highly contaminated, the risk for corrosion at a depth of 3 in. for 6-inch repair materials is not prevented for more than 28 years.

From the simulation results, a 10-inch concrete repair is more beneficial to the concrete contamination than a 6-inch concrete repair. A bigger repair provide more time before reaching the critical chloride concentration at a depth of 3 in. in the repair materials. However, the simulation results gave 36 years before reaching this value. Thus, it is recommended to include the presence of a corrosion inhibitor in the concrete repair materials. A dosage of 1 gal/yd^3 of a calcium nitrite corrosion inhibitor should be sufficient. At 4 in. inside the repair materials, the critical chloride concentration should be reached after more than 50 years *without corrosion inhibitor.*

Finally, it should be noted that the exposition in the tidal zone (above the low water level) is as severe as the exposition in the immersed zone. Thus, the concrete in the tidal zone should be considered as contaminated by chloride as the immersed concrete. The corrosion should propagate at a higher rate in the tidal zone where the oxygen is available. If on-site observations confirm the degradation of concrete in the tidal zone, the simulation of the repair options would be applicable for the concrete in this area. Moreover, it is more appropriate to repair with mixture 2 since its mechanical properties are similar to the in-place concrete. In all cases, the repair concrete should be properly air entrained to resist the effects of freezing and thawing in saturated conditions with or without deicer salt scaling.

MATERIALS SERVICE LIFE (MSL), LLC

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Project Manager

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Director of operations

Petrograhic examination performed by

Niels Thaulow
Technical Director, Construction Materials

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Appendix A
Core Pictures

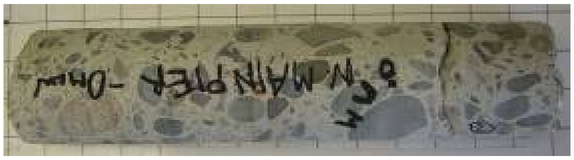

Figure A1 - core # 1-1

Figure A2 – core # 1-2

Figure A3 - core # 1-3

Figure A4 - core # 2-1

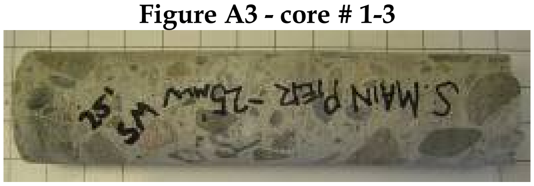

Figure A5 - core # 2-2

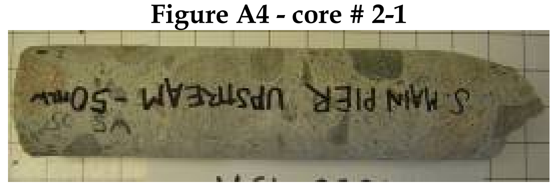

Figure A6 - core # 2-3

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Appendix B
Petrographic examinations

Sample 3638015
North Pier of Memorial Bridge
1-1

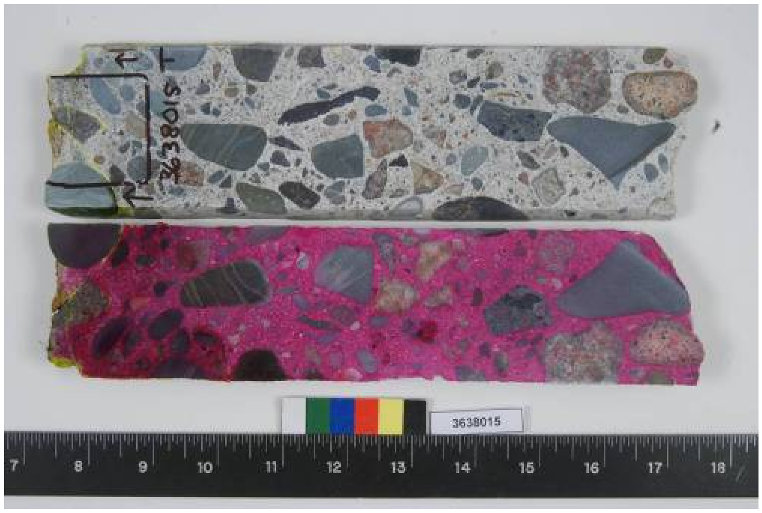


Figure B1 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015. Prior to cutting, the top exposed surface (left side of image) was impregnated with fluorescent-dyed epoxy to preserve the surface fractures. The top section of the half core was cut perpendicular to exposed surface. The thin section was taken from the upper face at the exposed surface. The lower cut face was treated with the pH indicator, phenolphthalein, but the epoxy at the surface has interfered with the test.

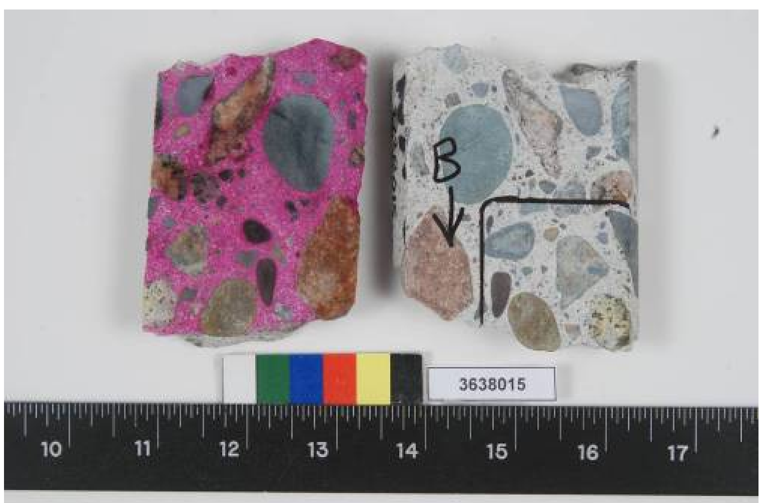
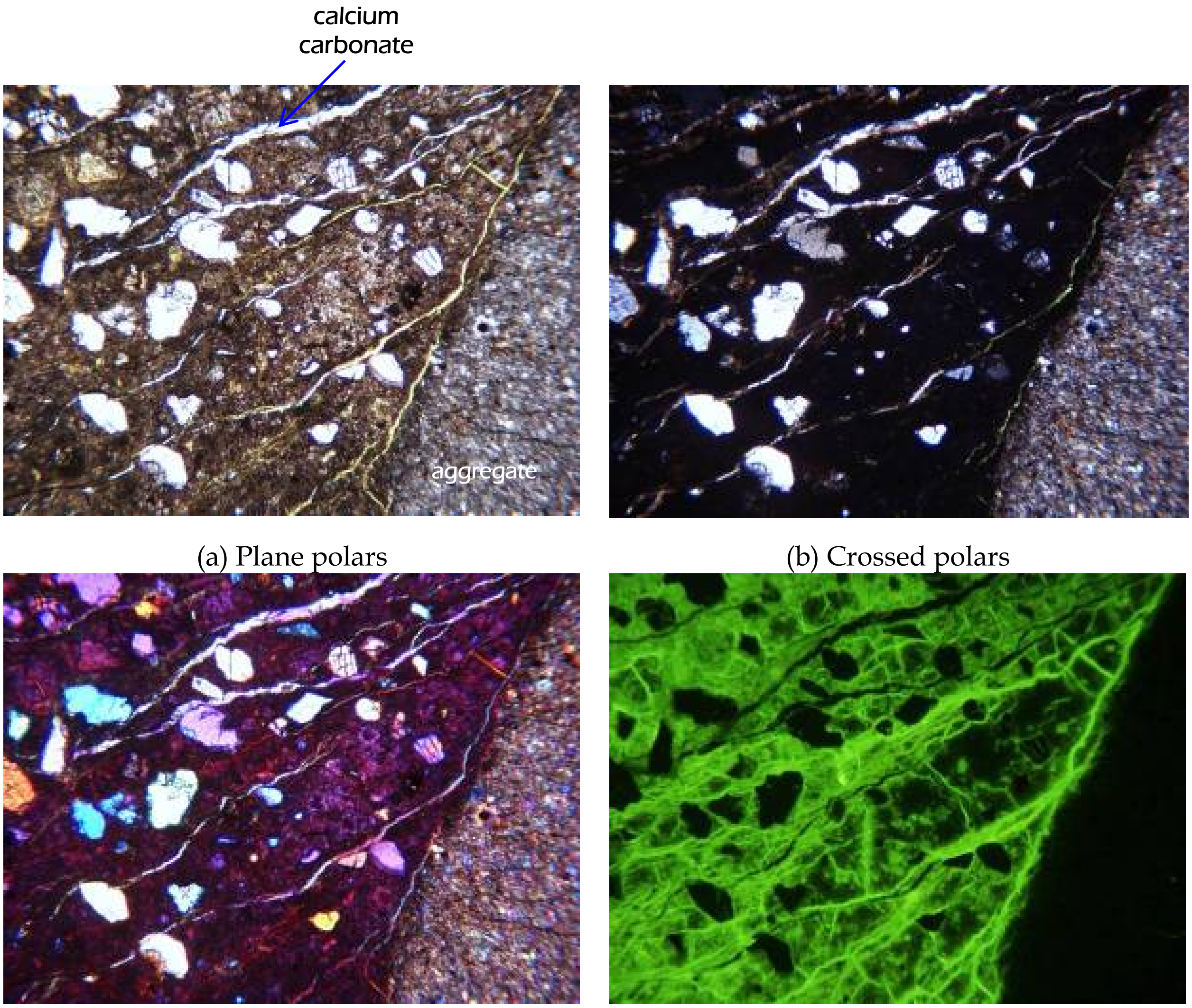


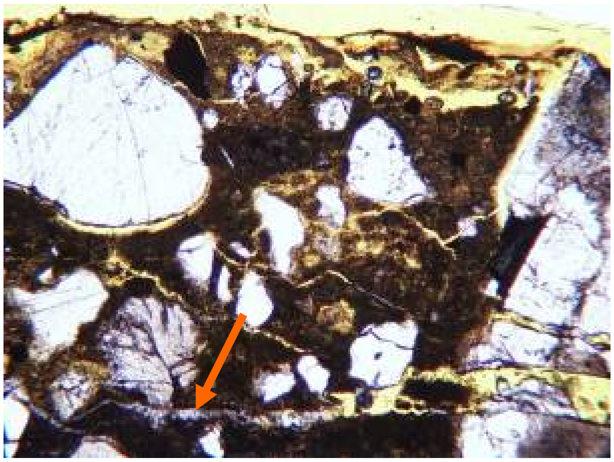
Figure B2 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015. The bottom section of the core was cut perpendicular to the exposed surface. The thin section was taken from the right face. The left cut face was treated with the pH indicator, phenolphthalein.

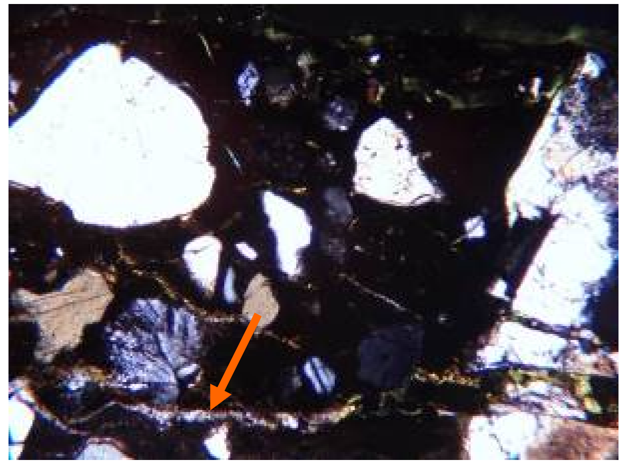


(a) Plane polars (b) Crossed polars

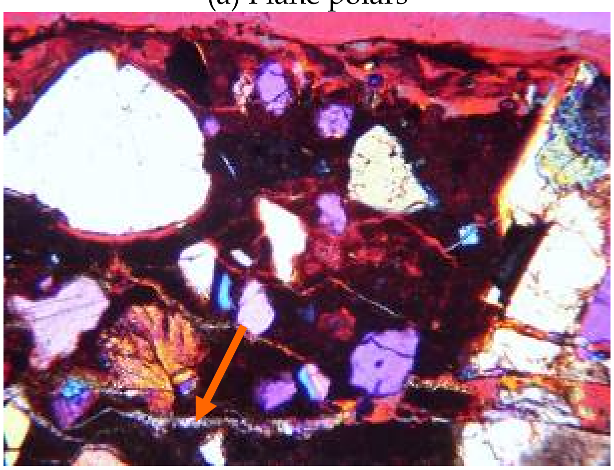
(c) Gypsum plate (d) Fluorescent mode

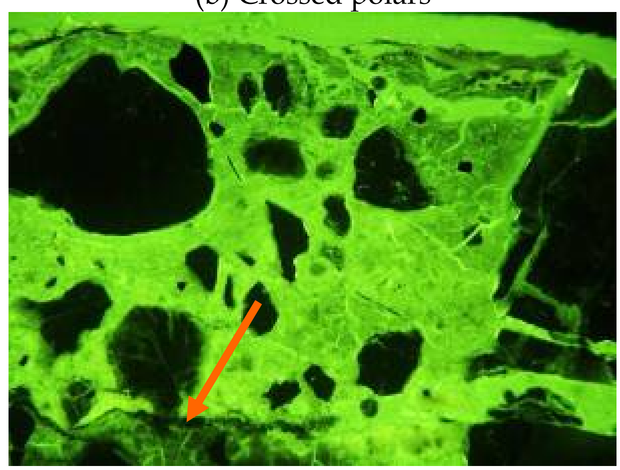
Figure B3 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. Optical images in different light modes showing cracks parallel to exposed surface in carbonated paste. Some cracks are filled with calcium carbonate which appears white in plane polarized light. Image area is 2.6 mm wide.





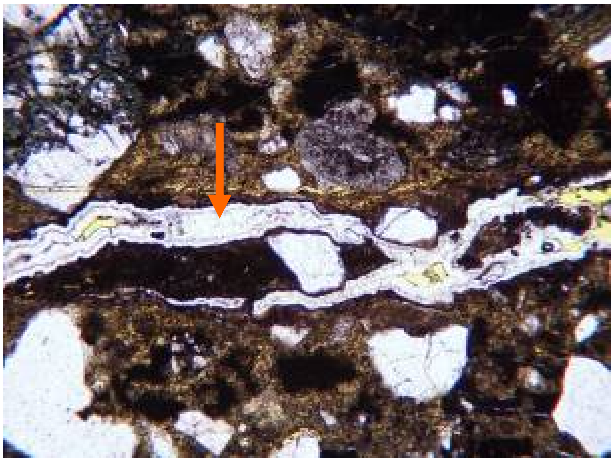
(a) Plane polars (b) Crossed polars

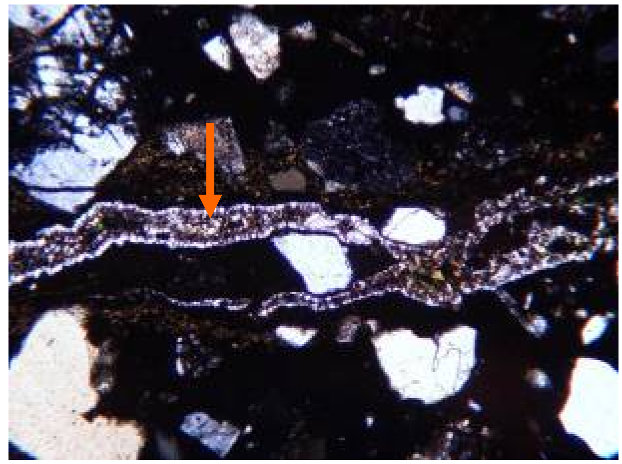




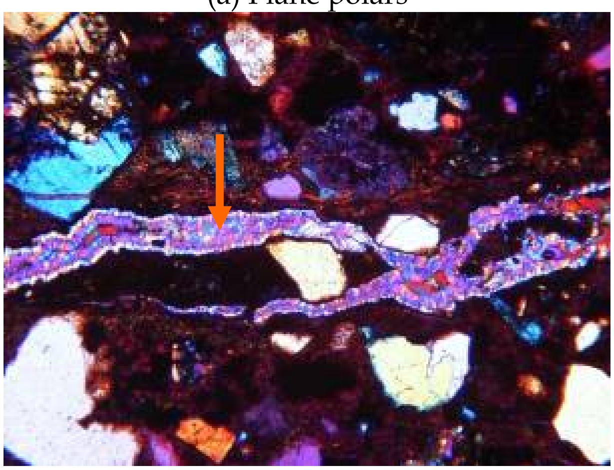
(c) Gypsum plate (d) Fluorescent mode

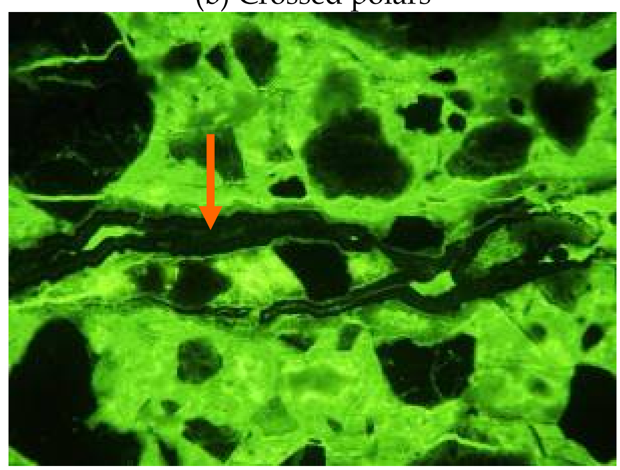
Figure B4 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. Optical images in different light modes showing cracks parallel to exposed surface in carbonated paste. Some cracks are filled with calcium carbonate; an example is indicated by an orange arrow in each image. Image area is 2.6 mm wide.





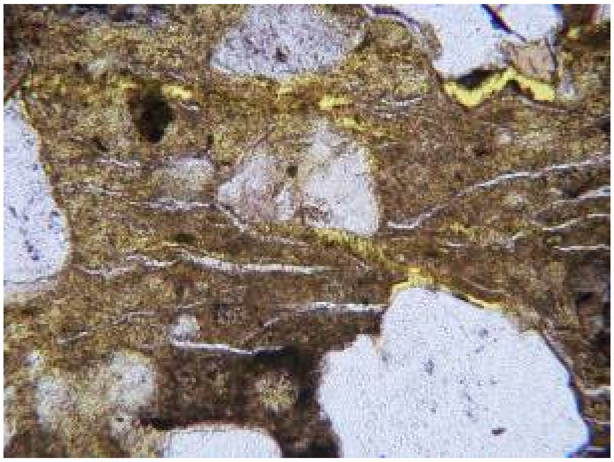
(a) Plane polars (b) Crossed polars



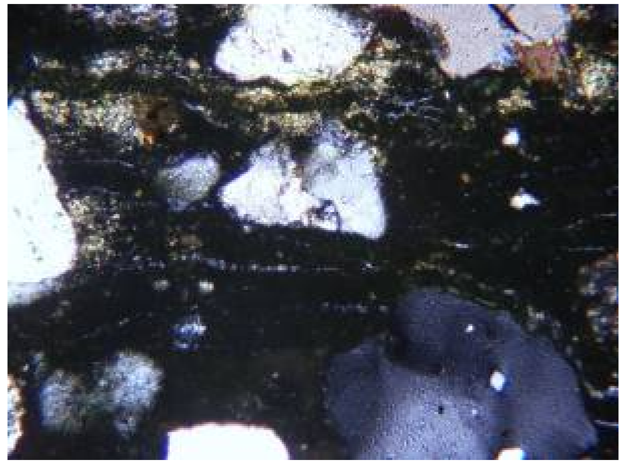


(c) Gypsum plate (d) Fluorescent mode

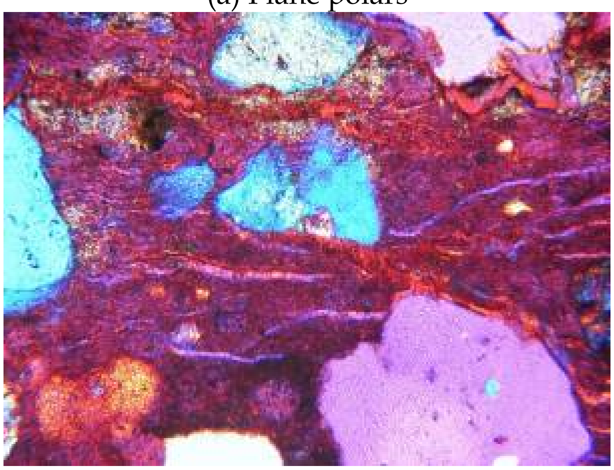
Figure B5 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. Optical images in different light modes showing magnesium hydroxide in cracks as indicated by an orange arrow in each image. Image area is 2.6 mm wide.



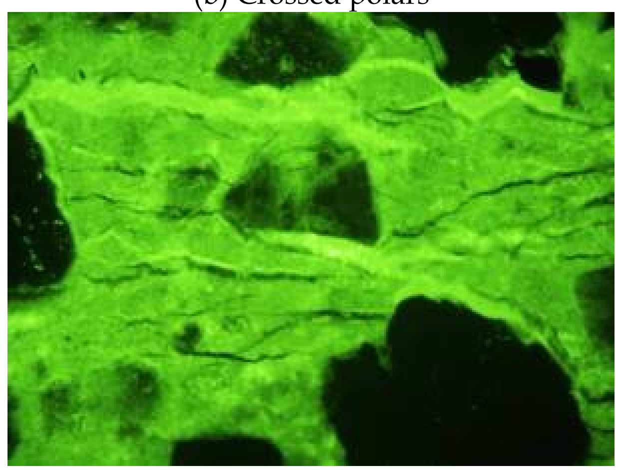
(a) Plane polars



(b) Crossed polars

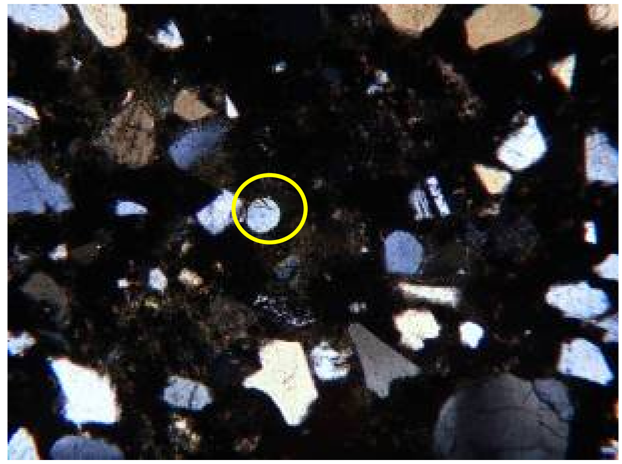


(c) Gypsum plate

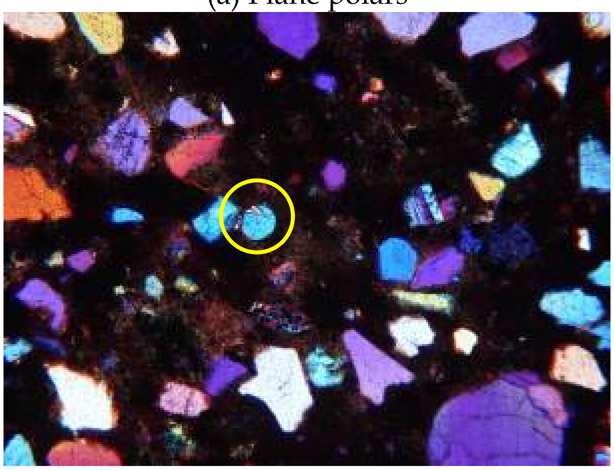
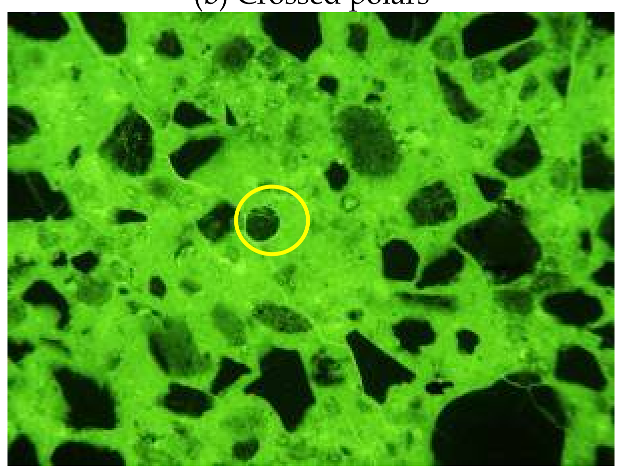


(d) Fluorescent mode

Figure B6 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. Optical images in different light modes showing parallel cracks lined with ettringite which appears white in plane polarized light. This field of view is also seen in Figure 8. Image area is 1.0 mm wide.

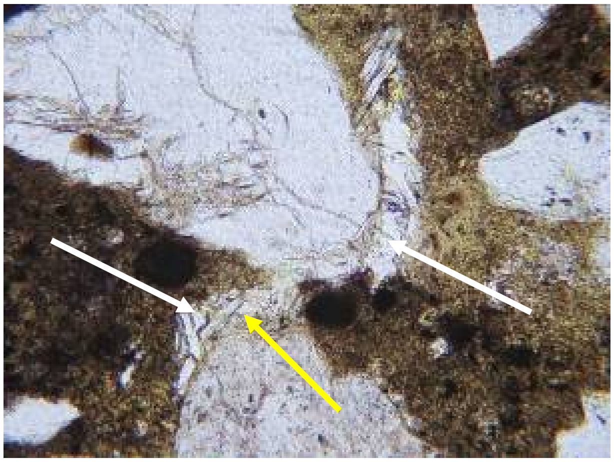



(a) Plane polars (b) Crossed polars

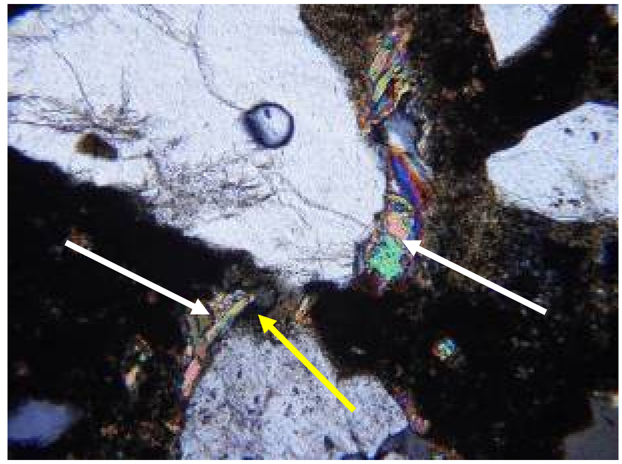



(c) Gypsum plate (d) Fluorescent mode

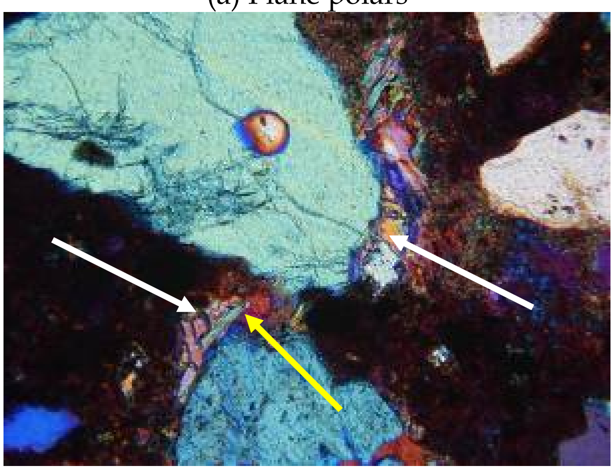
Figure B7 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015B. Optical images in different light modes showing a void filled with monosulfate (circled) in dense paste. Image area is 2.6 mm wide. The same field of view is seen in Figure B32.



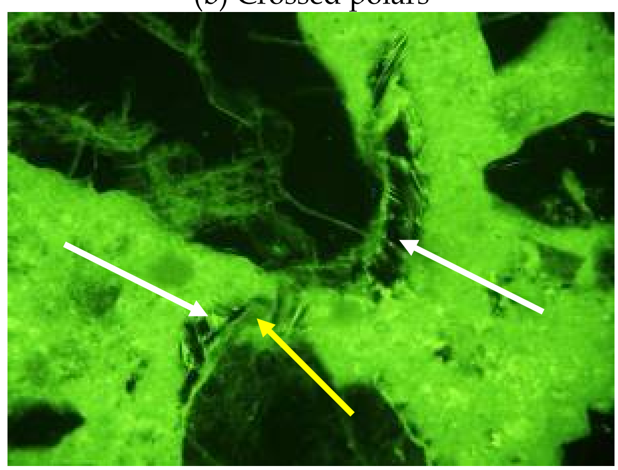
(a) Plane polars



(b) Crossed polars



(c) Gypsum plate



(d) Fluorescent mode

Figure B8 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015B. Optical images in different light modes showing monosulfate (yellow arrow) and calcium hydroxide (white arrows) in crack approximately 1' from exposed surface at water line. This same field of view is seen in Figure B34. Image area is 1.0 mm wide.

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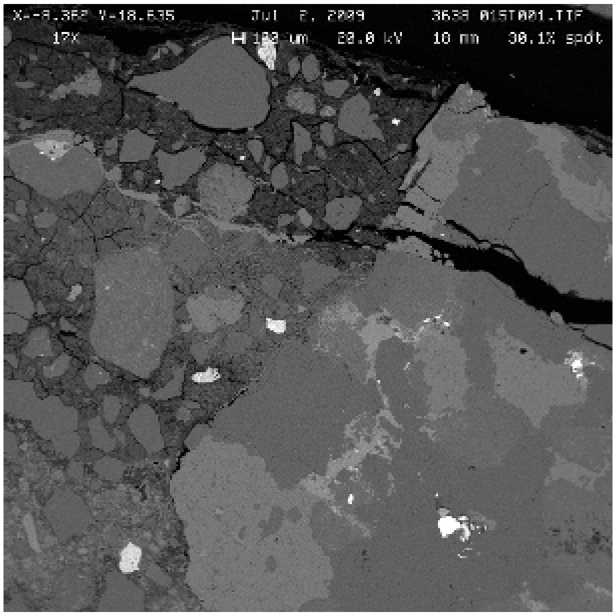

Figure B9 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM image showing top exposed surface with parallel cracks.

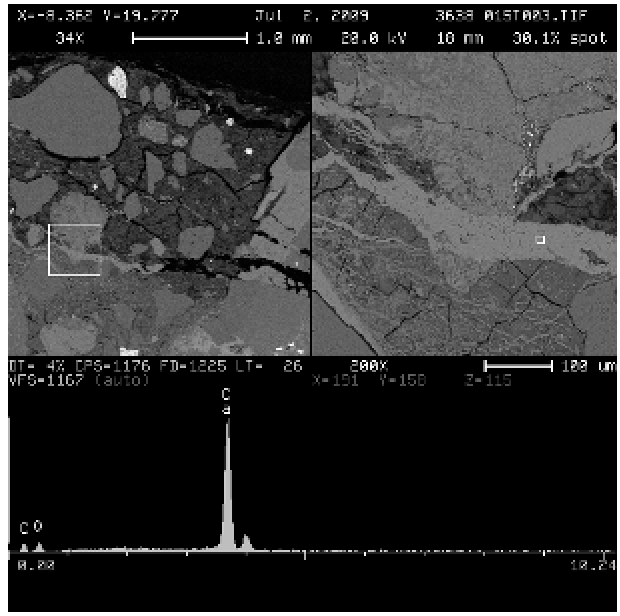

Figure B10 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing calcium carbonate in the crack.

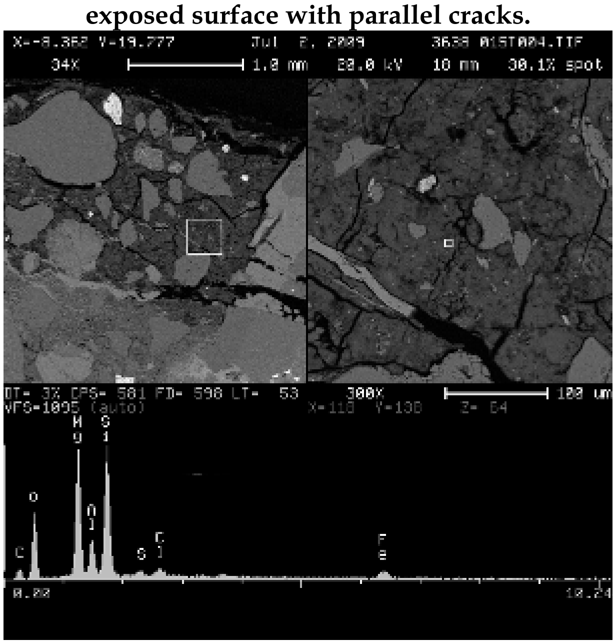

Figure B11 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing decalcified CSH converted to magnesium silicate with aluminum.

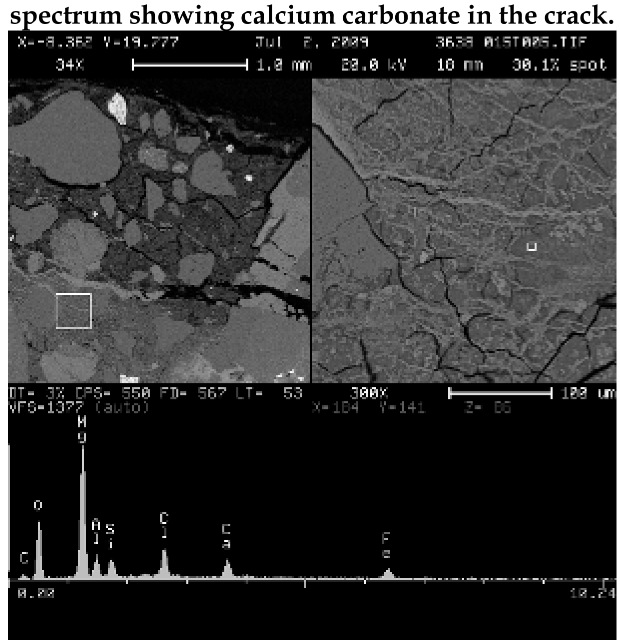

Figure B12 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing CSH rich in magnesium and chloride.

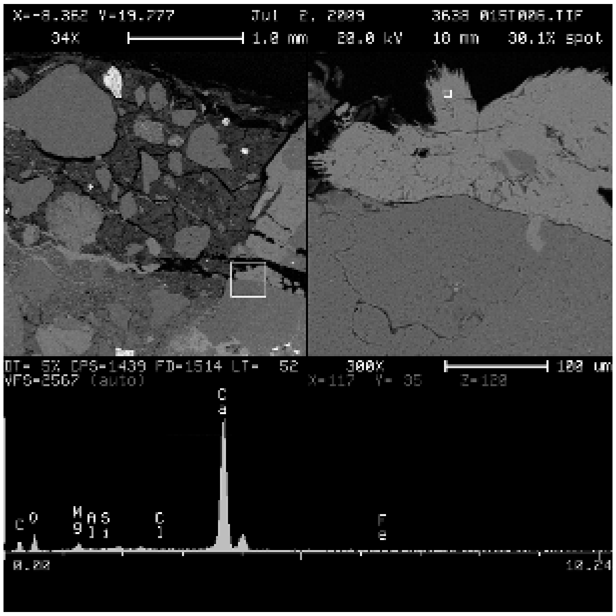

Figure B13 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing the composition of vaterite form of calcium carbonate.

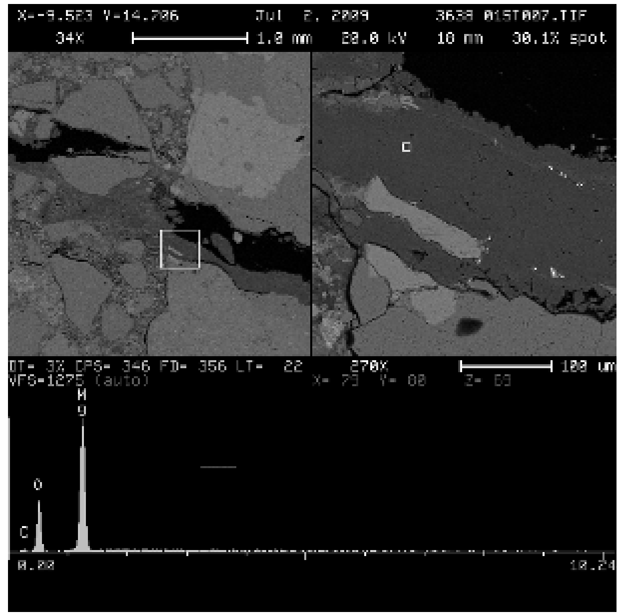

Figure B14 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing the composition of magnesium hydroxide (brucite) in crack.

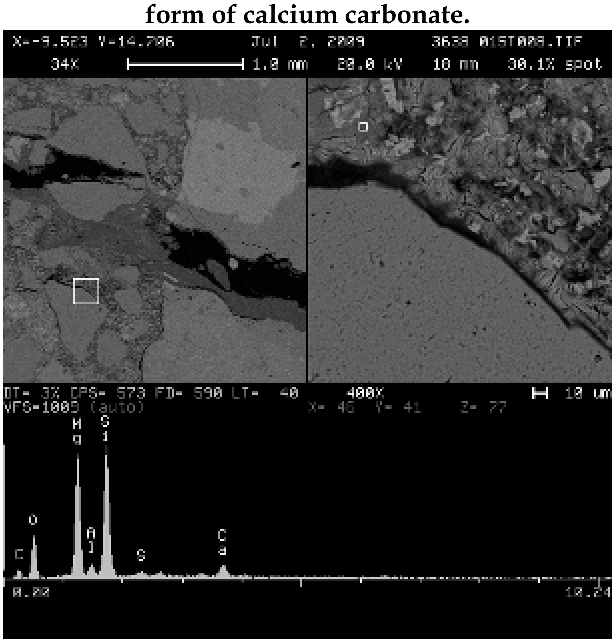

Figure B15 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing CSH converted to magnesium silicate with aluminum.

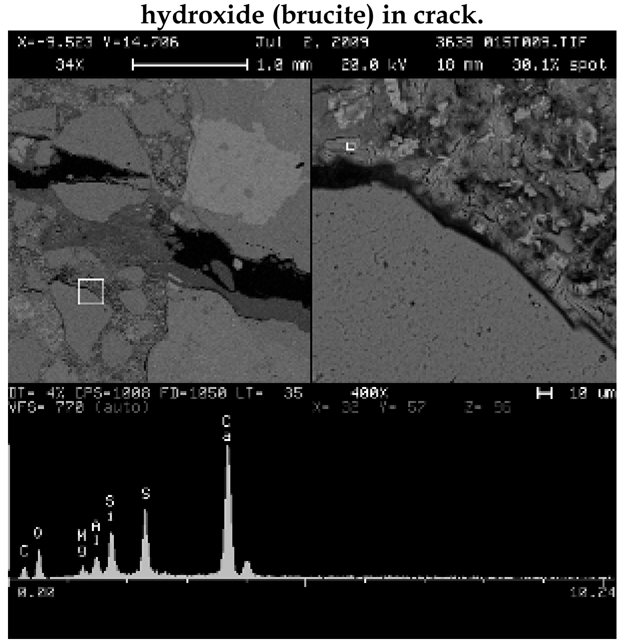

Figure B16 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing the composition of thaumasite. The majority of the material lining the crack is ettringite, as seen in Figure B17.

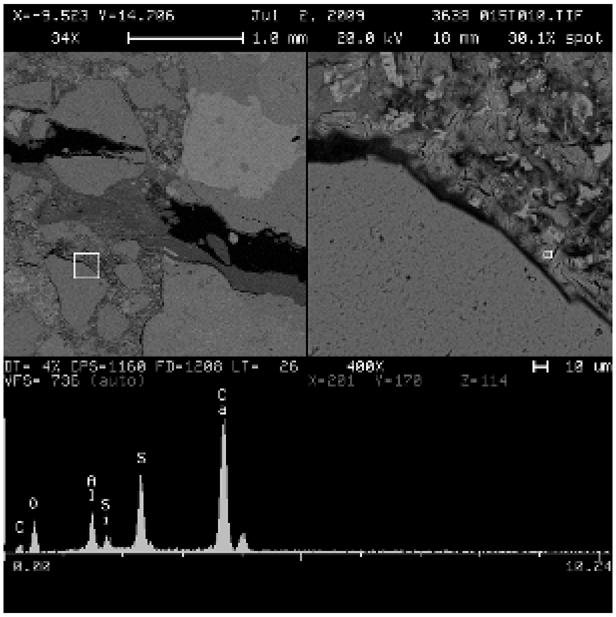

Figure B17 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing the composition of ettringite in a crack.

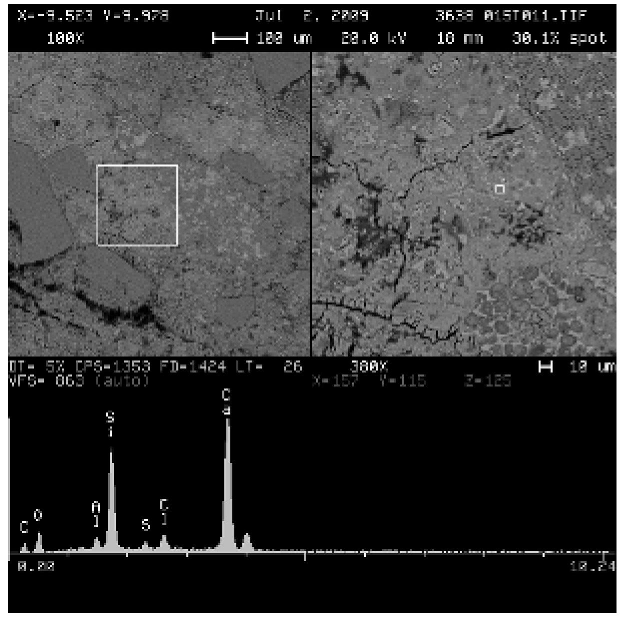

Figure B18 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing the composition of CSH with traces of sulfur and chloride.

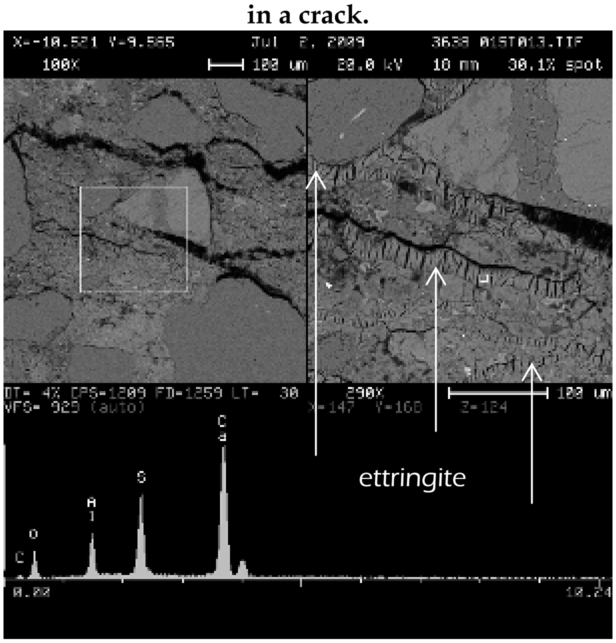

Figure B19 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing the composition of ettringite in crack.

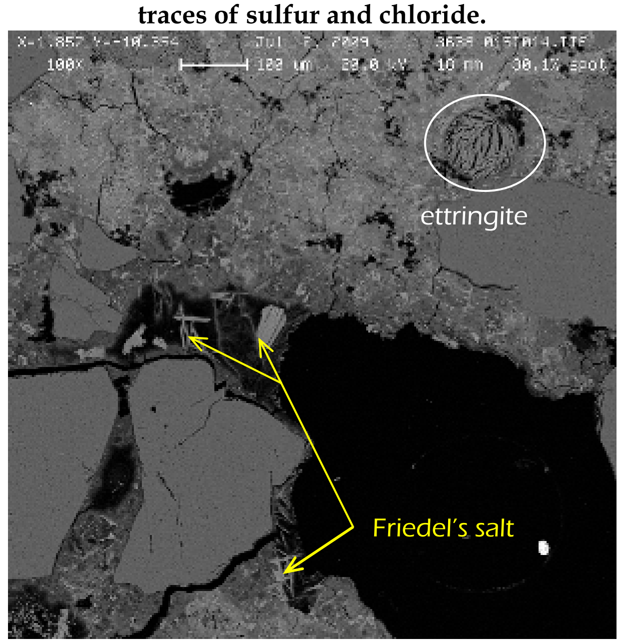

Figure B20 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM image showing Friedel's salt crystals and ettringite in an air void (near bottom of section).

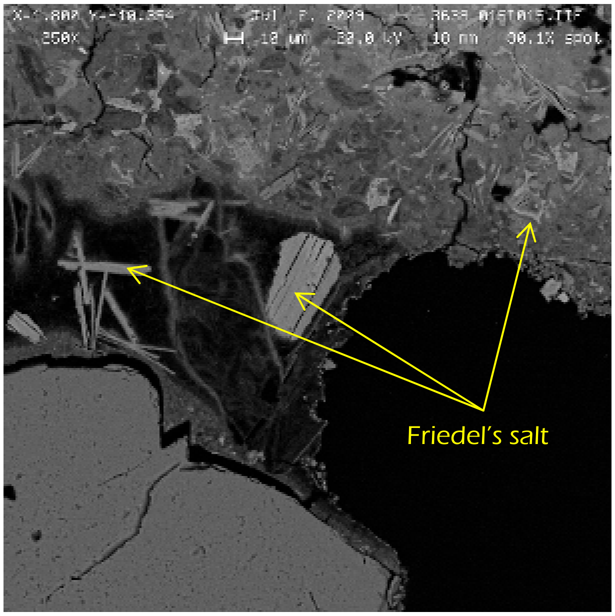

Figure B21 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM image showing Friedel's salt in crack (also smaller crystals in paste).

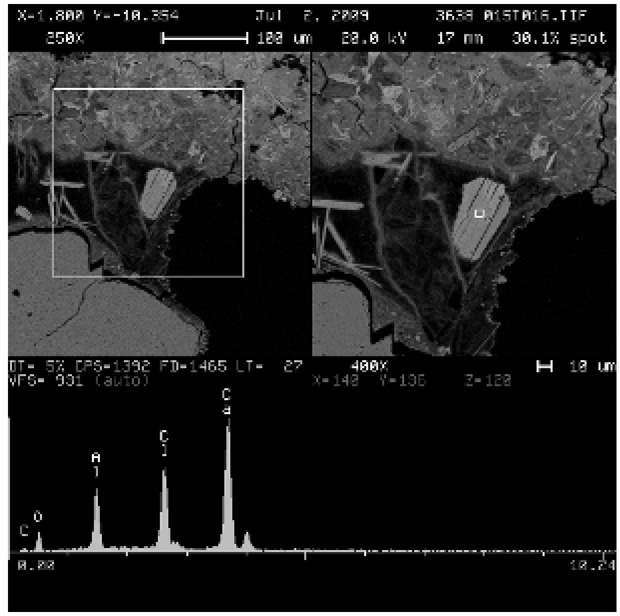

Figure B22 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing the composition of Friedel's salt.

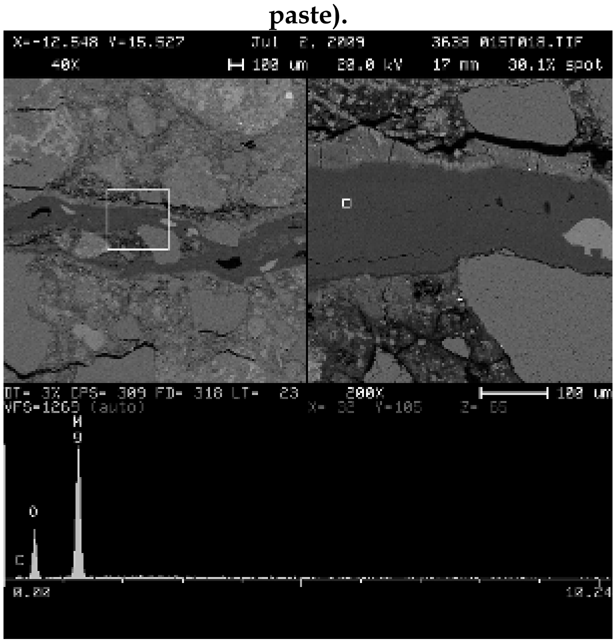

Figure B23 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing the composition of crystalline magnesium hydroxide. This is the same location as seen in Figure B5.

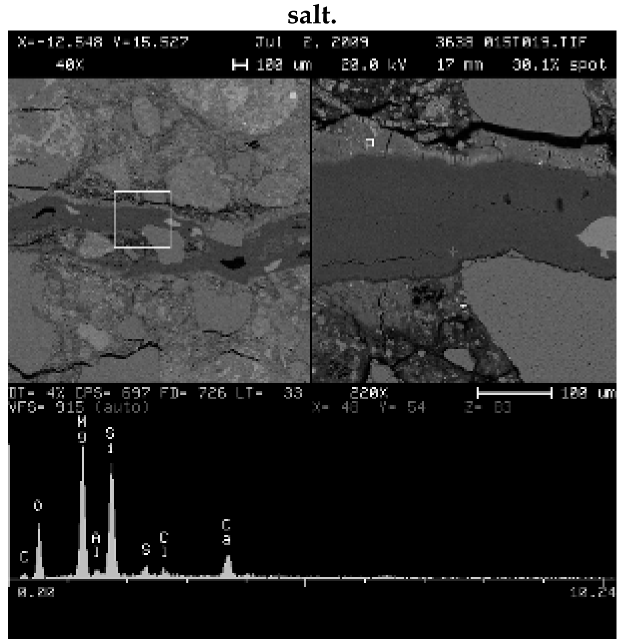

Figure B24 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing CSH converted to magnesium silicate with trace of aluminum. This is the same location as seen in Figure B5.

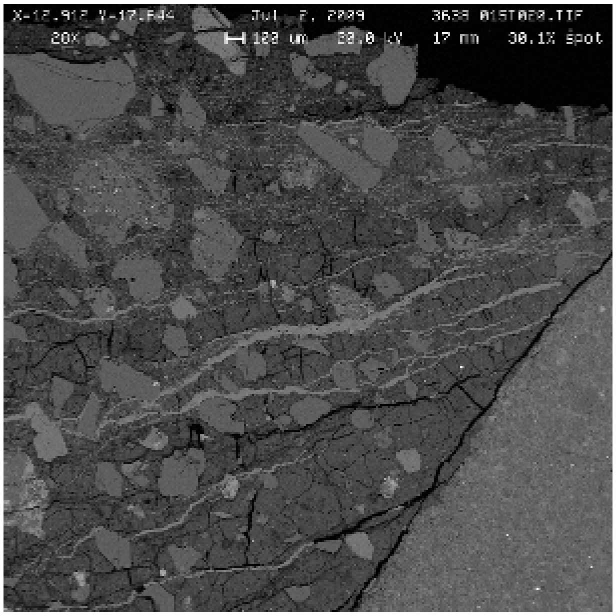

Figure B25 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM image showing parallel cracks at top exposed surface.

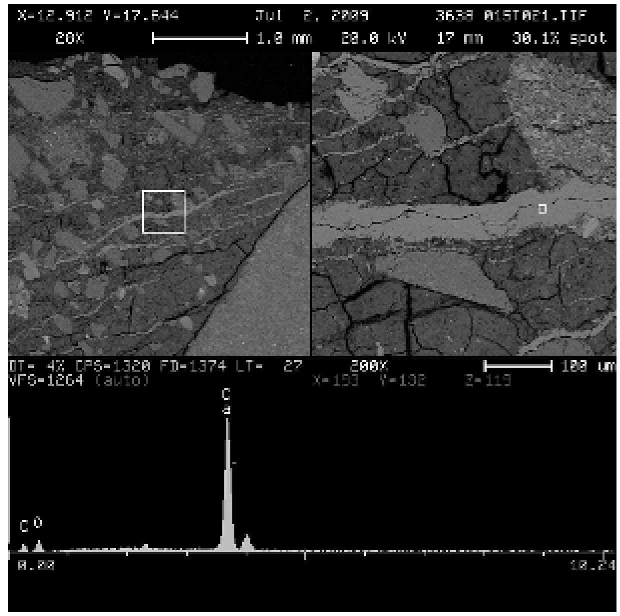

Figure B26 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing the composition of calcium carbonate in crack at the exposed surface.

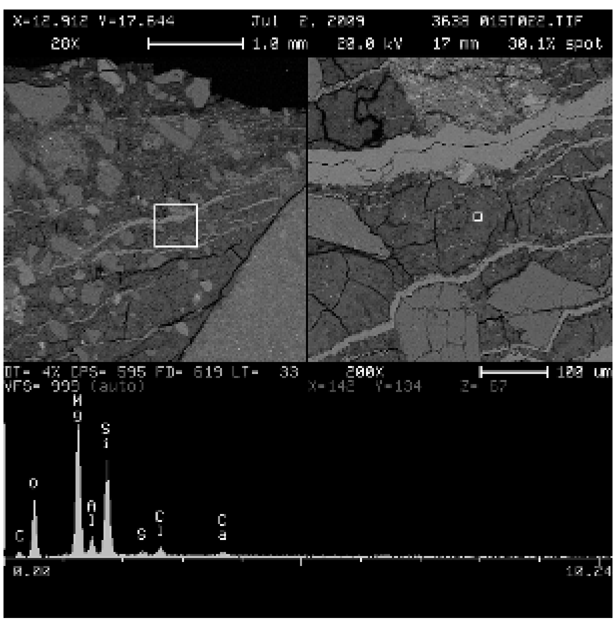

Figure B27 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015T. SEM images with EDS spectrum showing CSH converted to magnesium silicate with aluminum.

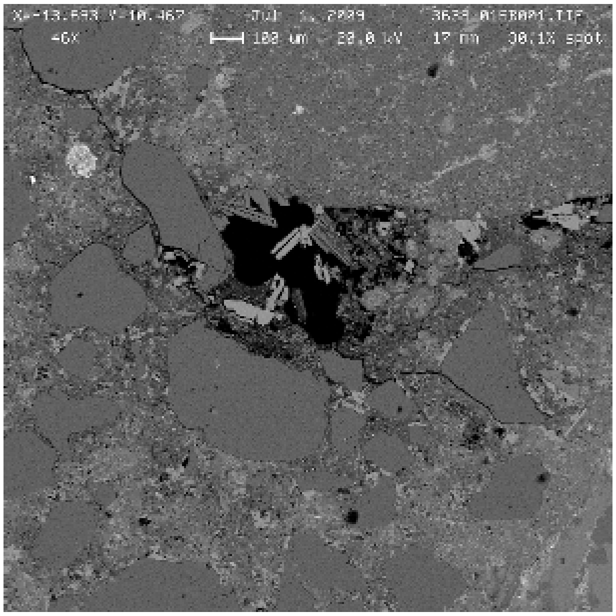

Figure B28 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015B. SEM image showing a partially filled air void.

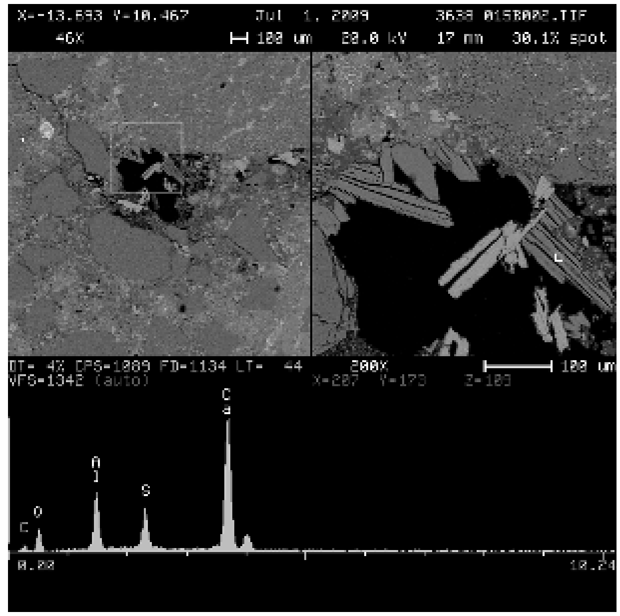

Figure B29 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015B. SEM images with EDS spectrum showing the composition of monosulfate partially filling the air void seen in Figure B28.

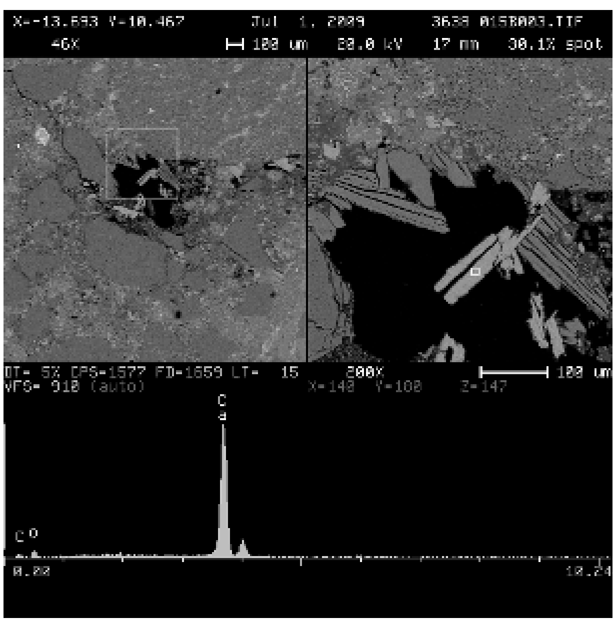

Figure B30 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015B. SEM images with EDS spectrum showing the composition of calcium hydroxide partially filling the air void seen in Figure B28.

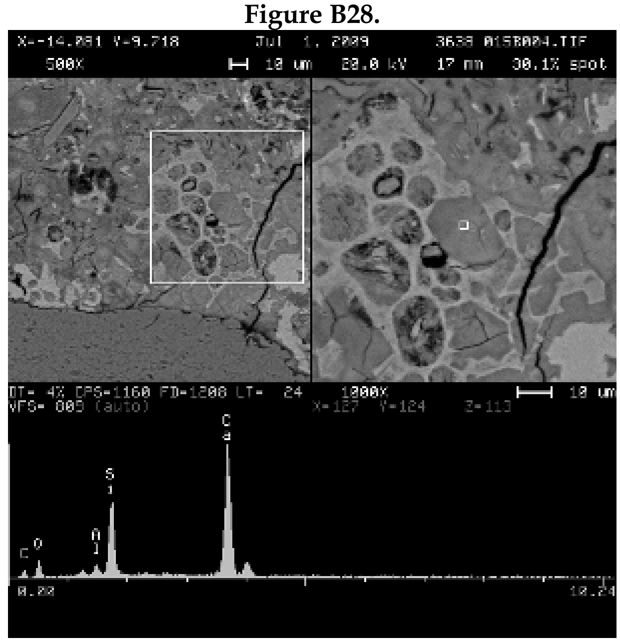

Figure B31 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015B. SEM images with EDS spectrum showing the composition of the CSH in a former cement grain.

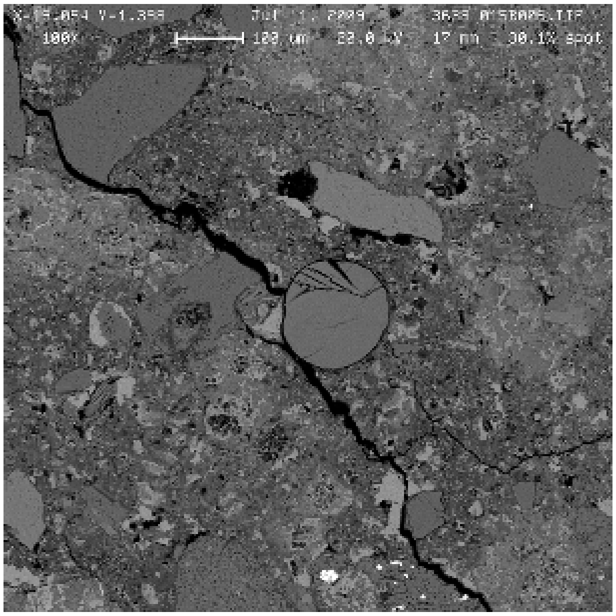

Figure B32 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015B. SEM image showing a close up of Figure B7 that contains an air void filled with monosulfate (circled).

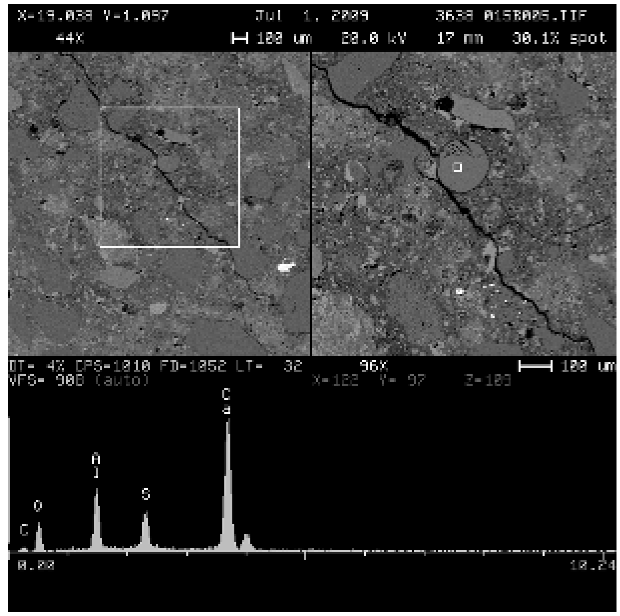

Figure B33 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015B. SEM images with EDS spectrum showing the composition of the material in the air void in Figure B32 as monosulfate.

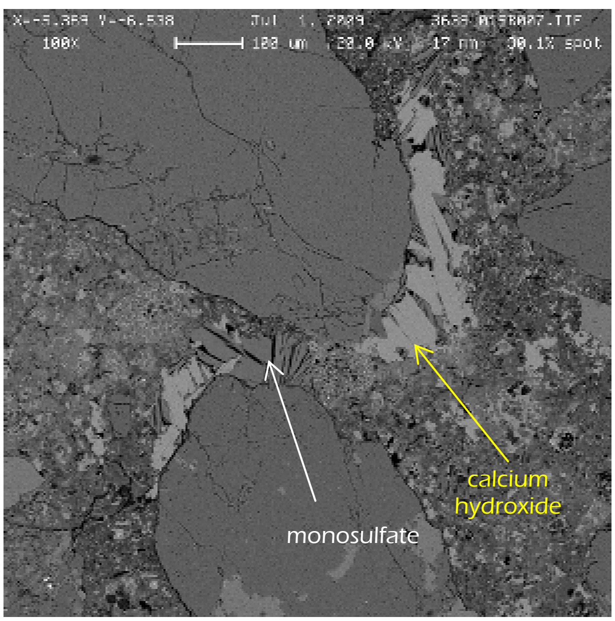

Figure B34 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015B. SEM image showing a void filled with monosulfate and calcium hydroxide.

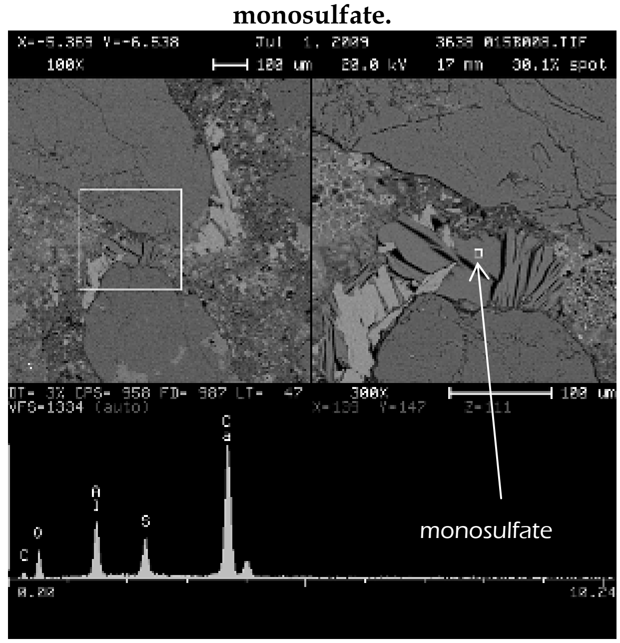

Figure B35 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015B. SEM images with EDS spectrum showing monosulfate in the void. The left-hand image is the same field of view as seen in Figure B34.

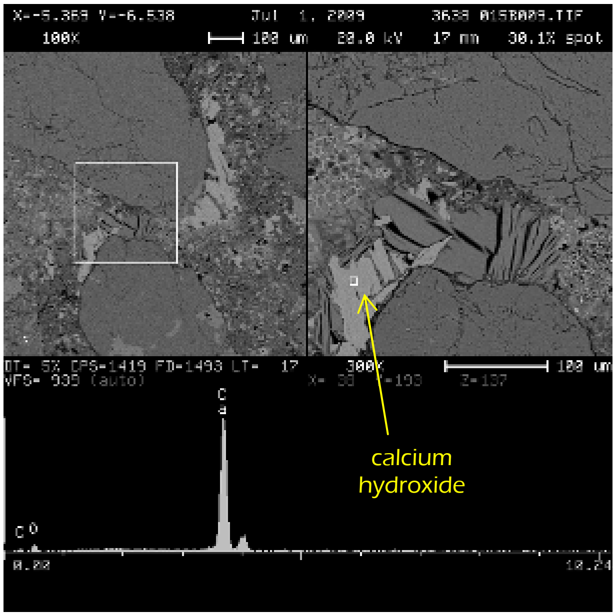

Figure B36 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015B. SEM images with EDS spectrum showing calcium hydroxide in an air void.

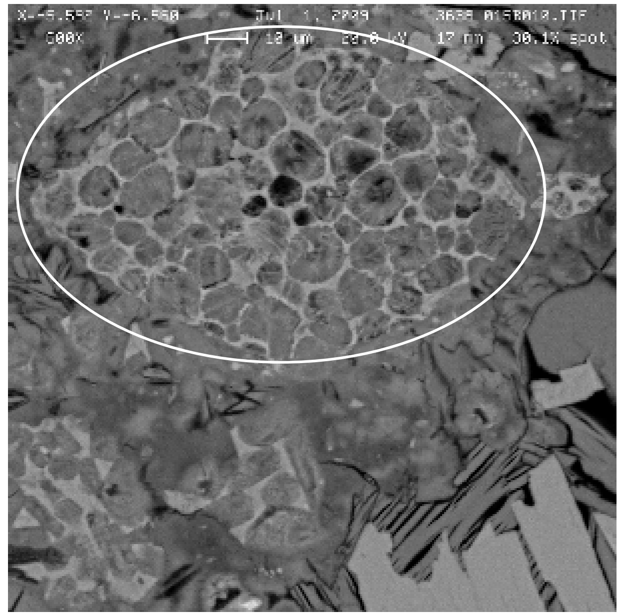

Figure B37 - North Pier of Memorial Bridge, 1-1, RJLG ID 3638015B. SEM image showing a fully-hydrated cement grain.

Paint Evaluation Report

COATING CONDITION ASSESSMENT OF PORTSMOUTH-KITTERY MEMORIAL BRIDGE PROJECT NO. 13678-E

Prepared For:

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Boston, MA 02111

Attn: Ms. Loretta Doughty, Structural Section Manager

Prepared By:

KTA-TATOR, INC.
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Robert Lanterman

Robert Lanterman
Project Professional

November 20, 2009

RSL:MPR:kmm
JN290673
HDR_ Memorial Bridge Report.doc

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INTRODUCTION

In accordance with an agreement between HDR Engineering, Inc. (HDR) and KTA-Tator, Inc. (KTA), KTA has completed a coating condition assessment of the Memorial Bridge for the Portsmouth-Kittery 13678-E Project. The assessment was performed by Robert Lanterman of KTA on September 29, 2009. The purpose of the assessment was to determine the condition of the existing coating systems applied to the structures. The following report includes the results of the field investigation, a discussion of coating rehabilitation options, and opinions of probable cost for repainting. Photographs of typical coating conditions observed are also included.

SUMMARY

The coating system applied to the bridge steel was in poor condition overall. Corrosion was observed throughout the structure. Stratified corrosion, pitting, and section loss was also observed. Based on the overall percent of visible corrosion and the adhesion test results, spot/zone repairs and overcoating would not be recommended. Total coating removal and replacement would be recommended for the entire bridge.

Laboratory testing reported detectable concentrations of lead present in the existing coatings on the bridge. The presence of lead will require the implementation of special safeguards to protect workers, the environment, and the public during any painting operations as well as the management of hazardous waste.

BACKGROUND

The Memorial Bridge is through truss lift bridge that carries US 1 across the Piscataqua River. The bridge is 1783’ in total length (including the Kittery approach) and was constructed between 1920 and 1923. The specific coating maintenance history was not available.

FIELD INVESTIGATION

Mr. Robert Lanterman of KTA performed the field coating condition assessment on September 29, 2009.

Access to the bridges was made available by Ms. Loretta Doughty of HDR, Inspection Team Leader Mr. Justin Stewart of HDR, and Mr. Gene Popien the Bridge Maintenance Manager for the New Hampshire DOT. The below deck portions were accessed at the piers. The lift span was fully raised to provide access to the lift span towers. Access to the Kittery approach was made from grade below the structure.

Testing Protocol

The following tests were performed in order to determine the condition of the existing coatings on the structure:

- Visual – A visual assessment of the coated surfaces was conducted to determine the extent, location, and any noticeable patterns of coating deterioration and/or corrosion. Assessments were made in general accordance with SSPC-VIS 2, Standard Method of Evaluating Degree of Rusting on Painted Steel Surfaces.

- Coating Thickness – Dry film thickness was determined using a Positector 6000. The Positector 6000 is a portable, battery operated, digital coating thickness gage, which non-destructively measures non-magnetic coating thickness over ferrous substrates using a magnetic principle. Calibration was verified prior to and after use with the National Institute of Standards and Technology (NIST) thickness standards.

- Number of Coats – The number of coats present and the thickness of each were determined using a Tooke Gage Mark IV with a 2X-cutting tip. This hand-held gage with a microscope (50X) destructively measures the thickness of each coat in multi-coat systems (up to 50 mils). Observation of a coating cross-section created with a cutting tip of known angle shows coating thickness and can be used to detect intercoat contamination, voids, underlying rust, mill scale, and pinholes.

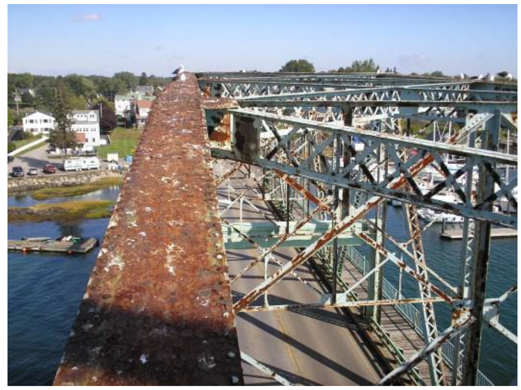
- Adhesion – Adhesion testing was conducted in general accordance with ASTM D3359, "Measuring Adhesion by Tape Test." Method A of this standard was utilized. Method A involves cutting an "X" through the coating down to the substrate using a razor knife, followed by the application of pressure sensitive tape (Permacel 99). The tape is then sharply removed from the X-cut and the amount of coating detached is then rated in accordance with the ASTM rating scale. Typical ratings of 4A to 5A are considered by KTA to represent good adhesion; 2A to 3A represent fair adhesion, while 0A to 1A represent poor adhesion.

- Photographs – Photographs of typical conditions were taken and are included in this report.

Visual Observations

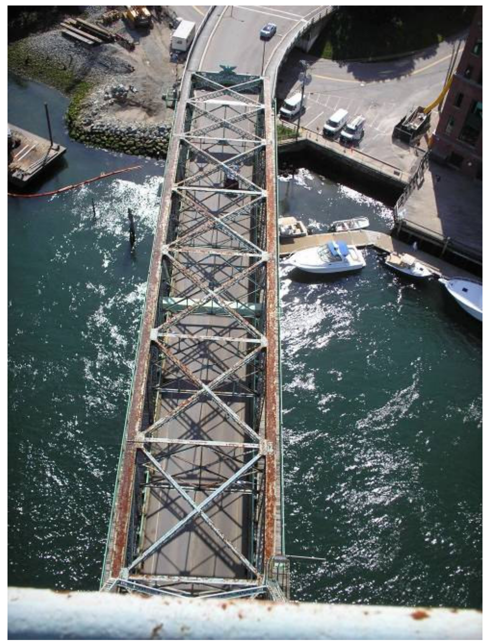
The coating system applied to the bridge steel was in poor condition. Approximately 50% corrosion was observed over the surfaces of the steel members. The above deck portions on the truss spans were in poor condition. Corrosion was observed in the splash zone as well as the upper truss members.

Photograph 1 – Top chord portal



Photograph 2 – Top surface of top chord

Photograph 3 – Top surface on top chord



Photograph 4 – Fixed truss portion

The interior portions of the box members were in poor condition, with the amount of corrosion similar to the exterior portions. Pack rust was observed at the lattice bar connections. Section loss was observed on some of the rivet heads.

Photograph 5 – Corrosion on lattice **Photograph 6 – Corrosion on interior portion**

The coating on the below deck members was in poor condition. Increased corrosion levels were observed under the open grating deck portions of the lift span. Areas of pack rust, pitting, and section loss were observed on the stringers and floor beams. Section loss was observed on some of the rivet heads and stratified corrosion was noted on the bearing pin nuts.

Photograph 7 – Below deck on fixed truss **Photograph 8 – Below deck on lift span**

Photograph 9 – Corrosion on stringer **Photograph 10 – Section loss at floor beam**

Photograph 11 – Section loss on rivets

Photograph 12 – Section loss on bearing pin nut

The lift span towers were in poor condition overall. Corrosion was observed on the tower members as well as the mechanical portions located on top of the towers.

Photograph 13 – Tower and counterweight

Photograph 14 – Tower and counterweight

Photograph 15 – Cable sheave

Photograph 16 – Cable sheave

The Kittery approach spans were in poor condition overall. Corrosion was observed to be increased on the fascia members and on the steel members adjacent to the deck drains.

Photograph 17 – Kittery approach

Photograph 18 – Fascia corrosion

Photograph 19 – Deck drain

Photograph 20 – Abutment bearing

Total Dry Film Thickness (Positector)

Approximately 50 dry film thickness measurements of the total coating system were taken throughout the accessible portions of the bridge. Thickness measurements ranged from 6 mils to 27 mils with an average thickness of 14 mils.

Individual Coating Thickness (Tooke Gage)

Tooke gage tests were conducted on several bridge members. Field measurements consistently found that 6 coats of paint were present in the test areas. The prime coat was orange in color and 2 to 3 mils in thickness. The intermediate coats were green in color and 3 to 4 mils in thickness, orange in color and 2 mils in thickness, green in color and 2 mils in thickness, and orange in color and 2 mils in thickness. The finish coat was green in color and 2 to 3 mils in thickness.

Adhesion

Approximately 15 adhesion tests were conducted on the accessible bridge surfaces. The coating in the areas tested was visibly intact. Adhesion ranged from 0A (poor) to 1A (poor), with an average rating of 1A (poor). Forced detachment during the testing occurred primarily as a cohesive break within the prime coat.

Substrate Examination

The substrate was available for examination at all Tooke gauge and adhesion test areas. The substrate was covered with a layer of mill scale in the test areas.

Samples

The following five (5) coating samples were removed from the bridge for laboratory testing.

Table 1 – Sample Locations

Sample No.	Member	Location
7	Floor beam	Below deck
8	Truss diagonal	Fixed truss span
9	Sheave guard	NH Tower
12	Truss vertical	Lift span, splash zone
13	Floor beam	At expansion joint

LABORATORY INVESTIGATION

The laboratory investigation consisted of atomic absorption spectroscopy and ion chromatography. The methods of analysis and results of the investigation are provided below.

Atomic Absorption Spectroscopy

The Memorial Bridge Samples 7, 8, and 9 were analyzed for total lead, cadmium and chromium in accordance with AOAC Method 974.02. Briefly, this method entails digesting samples in acid, filtering and analyzing by flame atomic absorption spectroscopy. Results of the testing are presented in Table 2, "Toxic Metal Content."

Table 2 – Toxic Metal Content

Sample No.	Lead Content (ppm)	Cadmium Content (ppm)	Chromium Content (ppm)
7	285,738 (28.6%)	Non-Detectable	13,220 (1.3%)
8	251,290 (25.1%)	Non-Detectable	17,795 (1.8%)
9	236,570 (23.7%)	Non-Detectable	28,780 (2.9%)

Ion Chromatography

Ion chromatography (IC) was performed on two (2) coating chip samples (Samples 12 and 13). The samples were weighed and then gently boiled in deionized water for forty-five (45) minutes before being cooled and filtered. A Shimadzu Model LC-20AD pump, CCD-10A conductivity detector and a Shodex 424 anion column were used for the analysis. A water based buffer solution, as recommended for the Shodex 424 column, was pumped through the system at a flow rate of 1.0 milliliter per minute. A three (3) point calibration curve was established using standards of known quantities of six (6) anions, which are listed in Table 3, "Results of IC Analysis." Two (2) chromatograms are appended.

Table 3 – Results of IC Analysis

Sample No.	Chloride
12	373 µg/g
13	583 µg/g

DISCUSSION

General Discussion on Maintenance Painting

The purpose of this coating assessment was to assess the condition of the existing coatings on the structures and make recommendations for maintenance painting. Many factors affect the service life of a coating system. These include the type of coating originally applied, the type and quality of surface preparation, service environment, number of coats and film thickness, and the history of maintenance painting activities.

If a particular coating has provided satisfactory corrosion prevention and remains in relatively good condition, it is cost effective to extend the life of the system through overcoating, retaining as much of that original coating as possible. When the coatings are in poor condition, a "full removal" strategy is used, which removes all existing coatings. This strategy effectively places the bridge at the beginning of a new maintenance painting cycle. Little work will be required for at least 10 years, and then, it should involve only minor touch-up. This strategy, while safe and effective, is also expensive. A discussion of the various types of maintenance painting activities follows.

Maintenance painting options for bridge structures fall into four main categories: (1) deferral of maintenance, (2) spot repairs, (3) spot repairs with full overcoats, and (4) complete coating removal and replacement.

Each of these options is progressively more complex and requires progressively more work. Correspondingly, each option also offers greater long-term protection to the structure, but at additional costs. When paints containing hazardous metals are present, the issues associated with removing these paints impact the decision making process.

Deferral of Maintenance

Maintenance painting can be deferred if the existing coating system is in good condition, if the service life of the structure is limited, or there is some other benefit for postponing the work. If extensive corrosion is found and maintenance painting is deferred for a period of time, the level of surface preparation required to properly prepare the surface increases correspondingly, and if left unattended for too long, total removal will ultimately be required. In some cases, when the structure is corroding extensively but is still structurally sound, painting is deferred because the highest level of surface preparation (abrasive blast cleaning) is already needed, whether performed today or several years from now. The strategy in this case is to allocate the money to repair coatings on other structures that are not so badly deteriorated in order to stop the corrosion from propagating to the point that total removal is the only option for those structures as well.

Spot Repairs

Spot repairs, as the name suggests, involves surface preparation and coating application only to the individual spots of corrosion or coating breakdown. The amount of coating being removed is minimized, reducing the impact of hazardous materials handling, containment, and worker protection when toxic metals are present. Spot repairs also serve to repair the existing coating film only where it is needed, repairing the corroded areas and stopping the propagation of the breakdown. Coatings in essentially any condition may be spot repaired, but it is only practical when the level of breakdown is minor and somewhat isolated and covers a small percentage of the surface (e.g., 1 or 2%). A disadvantage of this approach involves aesthetics. The repair spots are clearly visible.

A variation of this type of localized repair includes zone or area repairs. This involves surface preparation and coating application over a larger area that exhibits more concentrated levels of breakdown, but the work is limited to those areas. For example, the bearing areas of girders are often zone painted on either side of an expansion joint, without any significant painting on the rest of the structure.

Spot Repairs with Full Overcoat(s)

The application of a full overcoat serves two primary purposes: the additional coat provides additional barrier protection and helps to seal minor defects that are not apparent when conducting spot repairs. It also offers an improved appearance when compared to spot repairs. The addition of the overcoat also adds complexity and cost to the overall project. The complexity increases because a contractor must now gain access to all areas of the structure to apply the full coat. The existing surface must also be thoroughly cleaned (i.e., power washed) to remove chalk and surface debris. The adhesion of the existing coating must also be good and sound; otherwise the stresses imparted by the overcoat can cause disbonding of the existing system, especially under freeze/thaw conditions. In some cases, two full overcoats are applied. This strategy is typically used when the amount of visible corrosion and coating deterioration covers less than 15% of the surface.

Total Coating Removal and Replacement

Total removal and replacement is the final option for maintenance painting. It is the most costly option (especially when removing existing coatings that contain toxic metals), but it offers the greatest opportunity for long-term protection. All of the mill scale, rust, and paint are completely removed and a new system with a new design life is applied. All paint containing toxic metals is removed at the same time, eliminating hazardous metals from future consideration. This method also provides the most pleasing appearance.

When total removal and replacement is performed, a new maintenance cycle begins. As the coatings age and weather, isolated spot repairs will be required. Several spot repairs may be made to the individual structure until a full overcoat is necessary. More spot repairs may then be made and additional overcoats applied until extensive corrosion develops, significant coating breakdown occurs, or the mechanical properties of the coatings (e.g., the adhesion) degrade to the point where additional work (spot touch-up or overcoating) is no longer practical. At this time, complete removal may again be required, but only after the maximum effective life of the original coating system has been extended through the planned maintenance activities.

RECOMMENDATIONS

The coating system applied to the bridge steel was in poor condition overall. Corrosion was observed throughout the structure. Stratified corrosion, pitting, and section loss was also observed. Based on the overall percent of visible corrosion and the adhesion test results, spot/zone repairs and overcoating would not be recommended. Total coating removal and replacement would be recommended for the entire bridge.

The specifications should require abrasive blast cleaning in accordance with SSPC-SP 10, "Near White Metal Blast Cleaning." The specifications need to recognize the presence of chlorides and include provisions for remediating them (e.g., pressure washing, steam cleaning, blast cleaning the steel and allowing it to rust overnight followed by reblast cleaning, using a mix of course and very fine abrasives, use of chemical chloride removal solutions, and others). Chloride contamination can be a particular problem for bridges in areas where de-icing salts are used on the road surfaces, especially in areas below expansion joints and open decks. Chloride must be removed for the surfaces prior to painting.

The recommended replacement coating system for these areas should involve three coats, consisting of an organic zinc rich prime coat, an epoxy intermediate coat, and a polyurethane finish coat, with stripe coats of the primer and intermediate coats applied to edges, crevices, rivets, and other irregular surfaces. This is the most common bridge coating system used across the country and is the most commonly tested system under AASHTO NTPEP (National Transportation Product Evaluation Program). Many epoxy zinc/epoxy/urethane systems are also approved under the NEPCOAT (North East Protective Coatings Committee) program.

Dealing with Lead

Laboratory testing reported detectable concentrations of lead present in the existing coatings on the bridge. The OSHA Lead in Construction Standard (29 CFR 1926.62) requires that controls be implemented if any detectable concentrations of lead are present. The OSHA Compliance Directive issued for the OSHA Lead in Construction Standard, Instruction CPL 2-2.58, states that if an employer has appropriately tested for lead (e.g., tested all layers of paints or coatings that may be disturbed) utilizing a valid detection method, and found no detectable levels of lead, then the standard does not apply. Paints with detectable concentrations of lead require the contractor performing the work to implement interim controls and assess actual employee exposures during the work in accordance 29 CFR 1926.62. Based on the lead results provided by the laboratory testing, 29 CFR 1926.62 is invoked during any activities that disturb the paint (e.g., abrasive blast cleaning, scraping, burning, and grinding).

It should be noted that other hazardous metals can also be present in the coating. Any disturbance of paint containing heavy metals in addition to lead must be performed in accordance with the requirements of the applicable OSHA standards.

In addition, containment will be required for the protection of the environment and the public, and the hazardous waste must be properly managed.

Caulking

Caulking should be considered for crevice areas, particularly those where pack rust has already formed. The caulking can be applied prior to the finish coat or after the finish coat. If applied before the finish coat, productivity can be affected as the caulking may take a few days to cure prior to painting. If applied after the finish coat, it will be visible and can pick up and retain dirt. A compromise is to apply the caulking after the finish coat and follow up with a brush coat of finish on the caulking a few days later to conceal it and to prevent dirt pick-up.

Cables

The lift span operates using a cable system. Measures to protect the cables during cleaning and painting must be taken, such as wrapping cables in a sheet rubber sleeve. A non-metallic abrasive may be preferred in these areas to prevent corrosion of any stray abrasive on the cables that may work its way under the protective shield. Any inaccessible areas where abrasive blasting can not be performed should be coated with a more surface tolerant system such as epoxy mastic or calcium sulfonate alkyd.

Wood Deck

The bridge has wood plank sidewalks along both sides of the bridge. The deck on top of the towers is also made of wood planking. Measures to protect the wood planking abrasive blast cleaning and painting must be taken.

Open Steel Grid Decking

The existing open steel bridge decking is in good condition overall. When blasting and painting members adjacent to and below the bridge decking, the preferred method is to remove the bridge decking while blasting and painting the support steel and reinstall the bridge decking when painting is complete. This allows blasting and painting for the portions of steel that would be below the bridge decking and since the bridge decking itself is in good condition it would not need to be painted. If the bridge decking remains in place, the edges will get blasted while blasting the support steel. These areas along the edges would require coating. Due to the good condition of the bridge decking, full blasting and painting of all of it would not be required.

Containment

The bridge crosses a navigable waterway. The lift span will have to remain operational during all maintenance work to allow boat traffic to pass. Containment of the lift span and towers will need to allow for raising the lift span.

OPINION OF PROBABLE COATING REPLACEMENT COSTS

A cost analysis was prepared for various options for maintaining the existing coating system. This analysis involved making various assumptions, based upon KTA and industry experience, of how a contractor might staff and proceed with the aforementioned recommendations. The site visit revealed limited space would be available to a painting contractor for equipment staging. Crew sizes, production rates, material and equipment requirements are evaluated and man-days and project-days are calculated. From the estimated project duration, costs associated with labor, materials, and equipment are factored in and the costs are developed. Overhead and profit are added as a multiplier to the base cost. For the purposes of this opinion of probable coating cost, labor was considered to be prevailing wage and equipment was calculated with rental rates.

Production days were calculated from the square footage of paintable steel surfaces and an allocated production rate. The surface areas for the bridges were performed from the provided drawings. The Memorial Bridge is estimated to be 306,700 square feet. The requirements for environmental protection, worker health and safety, waste disposal, and containment are included. Maintenance and protection of traffic during construction was not included in the costs. Finally, a variance multiplier is used on the final cost to develop a range of anticipated bid prices. This multiplier allows for the variations in contractor bidding techniques, new technology, and scheduling of the work within the painting season. The opinion of probable cost to perform total removal and replacement of the coating on the Memorial Bridge ranges from $4,550,000 to $5,500,000 for each. Total removal and replacement would take approximately 10 months total production for the bridge.

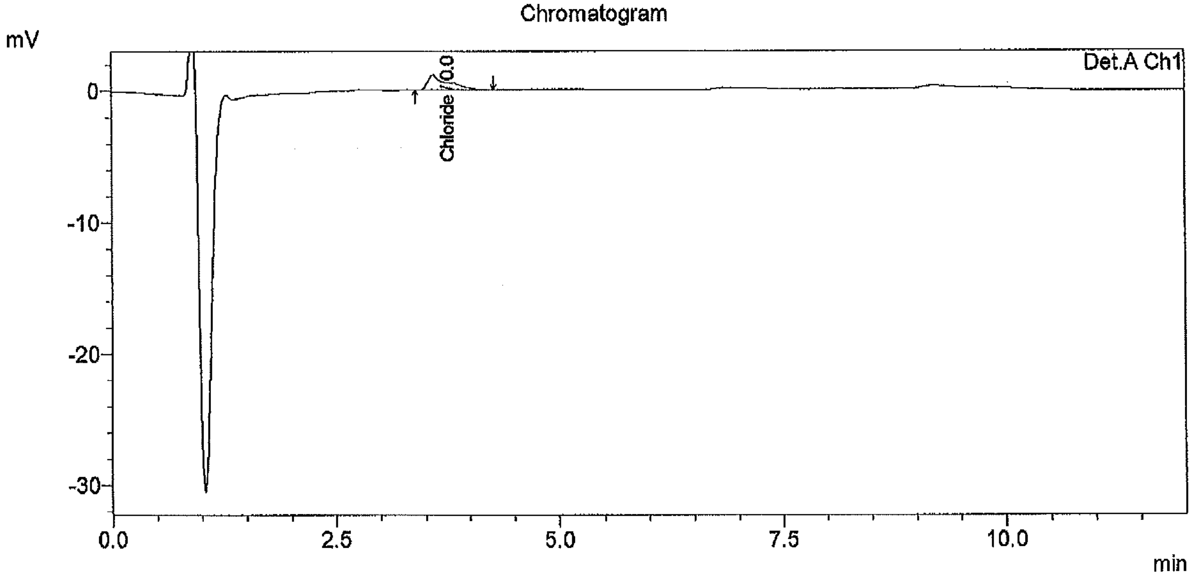
Opinions of probable construction costs are prepared on the basis of KTA's experience and qualifications and represent KTA's judgment as field professionals generally familiar with the industry. However, since KTA has no control over the cost of labor, materials, equipment, or services furnished by others, over contractor's methods of determining prices, or over competitive bidding of market conditions, KTA cannot and does not guarantee that proposals, bids, or actual construction costs will not vary from KTA's opinions of probable cost.

==== KTA Laboratory Ion Chromatography Report ====

Sample Information

Sample Name	: KTA-12 HDR
Sample ID	: UNK-0006
Injection Volume	: 25 uL
Data Filename	: 290673-4.lcd
Method Filename	: IC method.lcm
Report Filename	: IC report.lcr
Date Acquired	: 11/16/2009 12:55:34 PM
Data Processed	: 11/16/2009 1:07:36 PM

Chromatogram



1 Det.A Ch1/

Quantitative Results

Detector A

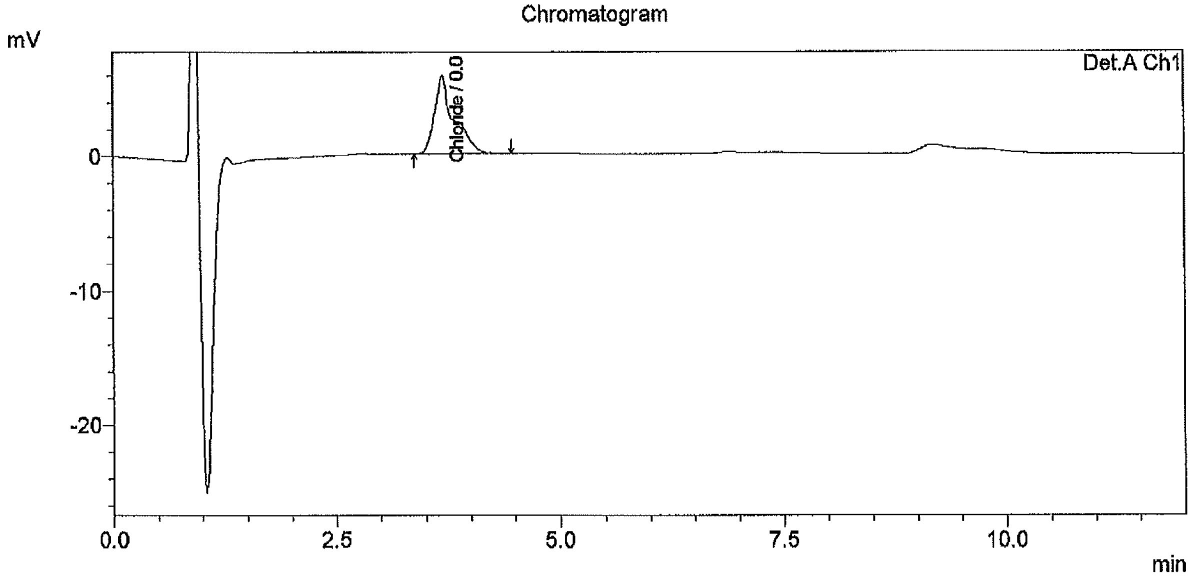
ID#	Name	Ret. Time	Area	Conc.
1	Phosphate	0.000	0	0.000
2	Chloride	3.597	17890	0.000
3	Nitrite	0.000	0	0.000
4	Bromide	0.000	0	0.000
5	Nitrate	0.000	0	0.000
6	Sulfate	0.000	0	0.000

==== KTA Laboratory Ion Chromatography Report ====

Sample Information

Sample Name	: KTA-13 HDR
Sample ID	: UNK-0007
Injection Volume	: 25 uL
Data Filename	: 290673-5.lcd
Method Filename	: IC method.lcm
Report Filename	: IC report.lcr
Date Acquired	: 11/16/2009 1:08:04 PM
Data Processed	: 11/16/2009 1:20:08 PM

Chromatogram



1 Det.A Ch1/

Quantitative Results

Detector A

ID#	Name	Ret. Time	Area	Conc.
1	Phosphate	0.000	0	0.000
2	Chloride	3.693	87792	0.000
3	Nitrite	0.000	0	0.000
4	Bromide	0.000	0	0.000
5	Nitrate	0.000	0	0.000
6	Sulfate	0.000	0	0.000

Mechanical & Electrical Inspection Report

New Hampshire Department of Transportation

MECHANICAL AND ELECTRICAL SCOPING STUDY FOR THE PORTSMOUTH MEMORIAL BRIDGE OVER THE PISCATAQUA RIVER

247/084

November 2009

PREPARED BY:

HDR Engineering, Inc.

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MECHANICAL STUDY

1 EXECUTIVE SUMMARY

A mechanical inspection of the Memorial Bridge was performed on May 27th, 28th, June 4th, and October 15th of 2009. A visual inspection of all main span mechanical systems was performed including all operating machinery, operating ropes, counterweight ropes, live load bearings and span guides. Measurements were taken on the gearing, deflector and counterweight sheaves, operating drums, operating ropes, and counterweight ropes as well as select counterweight rope tension measurements to assess the condition of the mechanical systems. The design alternatives presented attempt to improve the design of the bridge machinery and provide an expected useful operating life of 25 years. Three levels of rehabilitation scope were considered, described as follows: (1) Minimum Rehabilitation which address safety and reliability issues, and should be resolved in a timely manner, (2) Moderate Rehabilitation which will further increase the life and reliability of the mechanical systems. (3) Major Rehabilitation which provide the maximum level of reliability and safety as per modern design standards. Recommendations for each level of rehabilitation are presented in Section 5.

2 DESCRIPTION OF SYSTEMS

The Memorial Bridge is a span driven vertical lift span with single level deck. The bridge carries US Route 1 between the towns of Portsmouth, NH and Kittery, ME.

The operating machinery on the span consists of two main motors driving a central parallel shaft reducer. The parallel shaft gear box has two outputs which drive two sets of operating rope drums. The reducer is connected to each drum assembly via floating shafts. Thruster type motor brakes are mounted on each of the two input shafts of the reducer. Additional electro-magnetic disc brakes are mounted on shaft back stops opposite of the electric motor input shafts. A Liquid Propane Gas (LPG) backup motor is also mounted off of one of the electro-magnetic disc brake back stop shafts. The LPG motor is connected to the shaft via a clutch coupling and right angle gear reducer. The LPG motor is intended for emergency use when the electric motors are not available.

The operating rope drum assemblies consist of two operating drums driven by a pinion gear engaging a rack gear on each drum. The operating rope drums are wrapped with the up-haul and down-haul operating ropes. The operating rope drums rotate and play out or reel in the operating ropes during operation of the movable span. The opposite ends of the operating ropes are connected to fix points on the approach spans via ropes take-ups at roadway level and at the top of the towers.

The counterweight system for the movable span consists of two counterweights, one in each tower connected to the span via a series of counterweight ropes supported over sheaves mounted in the approach span towers. There are four counterweight rope sheaves, one in each corner of the movable span. The counterweight sheaves are supported on trunnion shafts by plain bearings. The counterweight rope connections at the counterweight have no means of adjusting the rope tensions and they are connected to the counterweight via a large equalizing device. The counterweight ropes are connected to the span via take-ups which allow adjustment of the rope tensions. Each counterweight has guides which engage rails along the length of the tower to ensure the position of the counterweight during travel.

The movable span has features in the form of span guides, and centering devices located on either end of the span to ensure proper position of the span while both moving and seated. The main movable span also has live load shoes in each corner to transfer the live load on the movable span to the rest piers.

3 INSPECTION APPROACH AND METHODOLOGY

A visual inspection of all main span mechanical systems was performed. This included operating machinery, operating rope system, counterweight rope system, live load bearings, and span guides. Various other measurements were taken to help assess the condition of the mechanical systems. Gear tooth thickness was measured using a gear tooth caliper and backlash was measured using feeler gages on all open gearing. Physical dimensions such as the rope diameter and lay length were measured using calipers on both the operating ropes and the counterweight ropes. Machinery bearing clearances and counterweight rope equalizer pin clearance were measured using feeler gages. Lastly, the rope tension in a random selection of counterweight ropes in each corner of the movable span was tested using the American Bridge method. The check of the counterweight rope tensions helped to assess the functionality of the counterweight rope equalizer.

4 INSPECTION FINDINGS

4.1 Operating Machinery:

In general the operating machinery is in good condition despite the age of the some of the components. With the exception of the operating drum racks and pinions, the machinery is in need of only minor to moderate attention. Despite these conditions it should still be noted that the age of the machinery makes it more susceptible to failure as well as reduces the likelihood of available replacement parts. The condition of other components of the movable span, such the condition of the movable span and the operating ropes also has a great effect on the course of action directed for the operating machinery.

4.1.1 Motors, Brakes, and Mounting:

The movable span is driven by two electric motors and braking is performed by a pair of thruster brakes and a pair of electro-magnetic disc brakes. All of these devices are in good condition. No mechanical rehabilitation work is suggested for the span drive motors. The motors shall be replaced in the moderate and major rehabilitation alternatives; further discussion is presented in Section 4.4 of Electrical Study. There would be some minor mechanical work related to the mounting and alignment of the motors in both the moderate and major alternatives. The following is the name plate information from the motors and brakes:

Westinghouse Motor

Style: 76F52585(North Motor), 76F525586 (South Motor)
Serial#: 1S-77
581-Z Frame
460 Volts, 163 Amps, 3Phase 60 Cycle
354 Rotor Volts, 141 Rotor Amps
Rated 100% load for 1 hour, 80°C rise
Max Values: 100 HP, 585 RPM, 2760 Ft.lb

Thruster Brake

General Electric Brake
Model: IC 9516-466T
Catalog: 011EB143
Max Torque: 1600 ft.lb
Min Torque: 60 ft.lb
Spring Length for 1600 ft.lb: 6 13/16”
230/460 Volts, 3 Phase 60 Cycles
1 7/8” Stroke

Electro-magnetic Disc Brakes

No Manufacturer’s Nameplate Information
Design Torque Rating Each: 700 ft.lb

Despite the good condition of the thruster brakes, it should be considered that the brakes are obsolete GE brakes and are no longer commercially available products (See photo M-1). Acquisition of spare or replacement parts may not be readily available in the event that any of the brake components fail. Minor work on the thruster brakes would consist of leaving the thruster brakes as is, considering their acceptable condition. The moderate and major work on the thruster brakes would be a complete replacement of the entire thruster brake with a new thruster brake of the same capacity.

The electro-magnetic disc brakes are in similar condition and situation as the thruster brakes (See photo M-2). The electro-magnetic disc brakes function properly but are of an outdated model and may suffer from a lack of availability of replacement parts. Another issue presented by the electro-magnetic disc brakes is that they are of a lesser capacity than the thruster brakes and bridge staff have indicated that they have had issues in the past related to the use of the electro-magnetic brakes in stopping the movable span.

The current brake arrangement also does not meet the requirements of the AASHTO movable bridge design code (2008 ed. 5.6.1) which states that mechanically operated bridges shall utilize two sets of brakes. One set, the motor brakes, shall be located on the motor input shaft. The other set of machinery brakes, shall be located as close to the operating ropes as possible. Ideally, one of the existing sets of brakes should be moved closer to the operating rope drums to be compliant with the recommendation of the AASHTO movable bridge design code. Practically, this course of action is not feasible given then existing machinery layout. A brake moved closer to the driving pinion would be of significantly greater capacity and cost. The available space in the machinery room also prohibits the placement of a significantly larger brake.

A similar course of action as the thruster brake is recommended for the electro-magnetic disc brakes. The minor and moderate rehabilitation alternatives assume no corrective action for the mechanical work on the electro-magnetic disc brakes. The major rehabilitation work assumes a complete replacement of the electro-magnetic disc brakes as part of the total replacement of the movable span.

All of the rehabilitation work on the motors and brakes need to be coordinated with the electrical rehabilitation work in Section 4, of Electrical Study.

4.1.2 Reducers and Back-up Motor:

There are two reducers in the operating machinery. The primary reducer is a quadruple input, double output parallel shaft reducer with the following name plate information:

Ratio: 31.4:1
235 HP @ 580 RPM

The primary reducer drives the operating rope drum and is driven by the electric motors and the LPG backup engine. The auxiliary reducer is a right angle, single input, single output reducer with the following nameplate information:

Ratio 11.4:1

101 HP @ 1600 RPM

The overall condition of both of these reducers is good and there are only a few minor maintenance issues which need to be addressed. Both reducers suffer from moisture intrusion into the interior of the housing (See photo M-3) and would benefit from the installation of a desiccant style breather. All rehabilitation alternatives with the exception of the major work alternative should include maintenance level work for the reducer. The major work alternative would include complete replacement of the reducers as part of the replacement of the entire movable span.

It should also be noted that during operation of the span, the backlash between the gear teeth would allow the gears inside the primary reducer to hammer against each other at an abrupt stop of the span. Although this is not a deficiency of the reducer, it is a deficiency of the operating machinery system and should be corrected to provide less abrupt stop of the span from the braking system. This corrective action would be considered incidental to the brake rehabilitation work, regardless of the alternative.

The Liquid Propane Gas (LPG) back-up motor and clutch was also visually inspected and observed during testing of the emergency drive. A clutch connects the LPG back-up motor to the auxiliary right angle reducer. Based on observation of the clutch and conversation with the bridge staff it was determined that the transmission leaks oil (See photo M-4). It was also noted that the clutch emitted a loud screeching noise during testing of the LPG back-up motor. Given the age of the LPG motor it is recommended that in the minor and moderate alternatives, the motor should be rebuilt. As part of the minor and moderate alternatives it is also suggested that the LPG back-up motor transmission and clutch be replaced. In the major alternative the LPG back-up motor and transmission will be replaced by other means of emergency operation power. Further discussion on the replacement of the LPG back-up motor is presented in Section 4 of Electrical Study.

4.1.3 Shafts and Couplings:

The shafting and couplings between the drive motors, gear reducers, and pinions are of various sizes and types. Overall all shafts and couplings are in good condition. Most shafts and couplings only have paint related deficiencies (See photo M-5). It should be noted that the motor couplings C1 and C2 are gear type couplings (See photo M-6) and do not meet the recommendation of the AASHTO movable bridge design code (2008 ed. 6.7.9.3). According to the code motor coupling are to be grid style coupling for the connection of motors to other machinery. Several additional instrument drive couplings were identified in the inspection which was not mentioned in previous documents, see appendix A for a machinery diagram showing the location and designation of these additional couplings. It was noted during inspection that the instrument drive coupling off the northwest operating drum, identified as coupling C-11 for the purpose of this report, may have a loose hub which does not fully engage the center spider of the coupling (See photo M-7).

Minor maintenance and painting are recommended for the minor rehabilitation alternative. It is recommended that the motor couplings be replaced with the appropriate grid style couplings in addition to the maintenance work for the moderate rehabilitation alternative. Finally, all couplings and shafts are to be replaced as part of the major work alternative.

4.1.4 Bearings:

The operating machinery bearings consist of a several sets of bearings in the operating drum frames that support the pinion shafts and operating drum shafts. These bearings are plain type bearing with split bushings and housings. In general the bearings have only minor maintenance related deficiencies; however several have wear beyond the standards for clearance (See photo M-8). A table of bearing clearance field measurements can be found in appendix C. It is important to maintain proper bearing clearances such that couplings and gear sets are not damaged by excessive shaft movement and vibration under load.

The minor and moderate rehabilitation alternatives for the bearings will require minor work to ensure the expected design life. The major rehabilitation alternative will include the complete replacement of the bearings.

4.1.5 Rack and Pinion:

The rack and pinions are used to drive the main movable span via the operating rope drums. Each pinion shaft drives two operating drums where each drum has its own rack gear mounted on the drum. The rack and pinions are the original installation on this bridge. The gear teeth show significant wear in all the open gear sets of the movable span (See photo M-9 &10). This poor condition of the gear teeth indicated that all of the rack and pinions are nearing the end of their service life and should be replaced. A table of gear tooth measurements can be found in appendix C. In all rehabilitation alternatives the rack and pinion gears should be replaced.

4.1.6 Operating Drums:

A single operating rope drum assembly consists of two operating drums wrapped with the up-haul and down-haul operating ropes. The operating drums were visually inspected and measurements of the rope grooves were performed to quantify the wear of the rope grooves. All exposed operating rope grooves were measured and found to have no appreciable wear. According to the plans the operating rope drums have a pitch diameter of 36 inches for a 1” diameter operating rope. This ratio of sheave pitch diameter to rope diameter is unacceptable according to the AASHTO movable bridge design code (2008 ed. 6.8.3.1.3). Further discussion on the impacts of the diameter ratio and its affects on the operating rope life will be discussed in section 4.2.3. Maintenance cleaning and painting are recommended for the minor and moderate rehabilitation alternatives. The operating rope drums will be replaced in the major rehabilitation alternative.

4.1.7 Instrument Drives:

The only externally mounted instrument drive is a chain and sprocket arrangement mounted off of the northwest operating drum shaft. The sprocket drive is in good condition and does not require any rehabilitation work. No rehabilitation work shall be performed on the instrument drives as part of the minor rehabilitation option. The instrument drive will be replaced in both the moderate and major rehabilitation alternatives.

Field limit switches are also located exterior to the operator house and operating machinery. Over travel and fully open limited switches are located in the towers and span seated limit switches are located on the rest piers. All of these switches are driven by the rehabilitation alternatives presented Section 4.8, of Electrical Study. No work shall be performed in association with the span seated limit switches. The over travel and fully open limit switches shall be replaced in all rehabilitation alternatives. The mechanical work to mount the new switches shall be included in all rehabilitation alternatives.

4.2 Operating Ropes:

The operating ropes are divided into two types of ropes, and are of the same construction. Uphaul ropes wind around the operating drums and are strung along the span supported by the operating ropes guides. The uphaul ropes bend around the deflector sheave and run up to the top of the tower spans where they are connected to the tower span by the operating rope take-ups. The downhaul ropes follow the same path as the uphaul ropes except that then bend around the deflector sheaves and run down to the roadway deck level and are connected to the tower span by operating rope take-ups. The operating rope system as a whole suffers from a variety of minor deficiencies throughout the system, but performs as intended. One of the major issues with the system is an outdated design which does not conform to current standards regarding rope and drum/sheave geometry.

4.2.1 Operating Ropes:

The operating ropes are 1" diameter Improved Plow Steel (IPS) 6x25 filler wire construction with a fiber core. Each rope has an open spelter socket at the end connected to the operating rope take-ups. The ropes were found to be in good condition with a few minor deficiencies related to corrosion and light wear identified during the inspection. According to past reports, some of the ropes are relatively new, having been replaced in 2001. It was noted that during operation of the span that the operating ropes rubbed against each other and even transposed position when the span was stopped and the operating ropes went slack. This condition shows that there is excessive slack in the wire ropes caused by the stretching of the operating ropes. The cause of the rope stretch shall be discussed further in section 4.2.3 in relation to the rope guides and sheaves. It was also noted that there was a kink in the East northeast uphaul rope just before the double rollers when the span was approximately 1/3 open position. The kink in the rope was visible when the load on the rope was relieved and the rope went slack (See photo M-11). Such a deficiency is grounds for replacement of the rope and the defect should be monitored while the wire rope is still in service.

In the minor rehabilitation work the operating ropes should have their tension checked and adjusted at the take-ups as needed. The moderate rehabilitation work includes the replacement of the kinked northeast uphaul rope in addition to the tensioning of the operating ropes. Major rehabilitation of the operating ropes would be complete replacement of the operating ropes as a result of the replacement of the entire movable span. This major rehabilitation would include bringing the operating rope system up to current standards.

4.2.2 Operating Ropes Take-ups:

The operating rope take-ups consist of a threaded rod with an eye on one end that is pinned to the open socket end of the operating rope. The threaded rod runs through a structural support on the tower and double nuts are used to adjust the tension on the rod. Generally the take-ups are exposed to elements and have varying levels of corrosion on their outer surface. No significant wear was noted on any of the take-ups. It is recommended that the take-ups be cleaned and painted in the minor work alternative. All other alternatives will have the take-ups replaced same-in-kind.

4.2.3 Guide Rollers and Deflector Sheaves:

Various size guide rollers and sheaves are used to support and guide the operating ropes along the truss of the movable span. In general these rollers and sheaves show signs of measurable wear and corrosion. In some cases, some of the rollers did not turn during the operation of the movables span and allowed the operating ropes to slide over them. Most of the intermediate deflector sheaves showed sign of wear in the rope grooves. These rollers showed further evidence of the poor operation in the form of groves and scoring over the outer diameter of the rollers caused by contact with the operating ropes (See photo M-12).

In addition to the known deficiencies with the rollers and sheaves there is an inherent design feature of these devices which does not meet the AASHTO movable bridge design code (2008 ed. 6.8.3.1.3). Similar to the design of the operating rope drums, the guide rollers and deflector sheaves have an unacceptable pitch diameter to rope diameter ratio. Guidelines for the diameter ratio are presented to reduce known stress and wear due to bending the wire ropes over the sheave and to increase the serviceable life of the rope. The original design of the drums, sheaves, and rollers has subjected the operating ropes to higher than recommended stress which has resulted in the premature elongation of the operating ropes. The elongation of the operating ropes is the cause of the excessive slack of the ropes witnessed in the operation of the movable span. The additional stress will also reduce the service life of the operating ropes.

As part of the minor rehabilitation of the guide rollers and deflector sheaves, all rollers should be replaced and all sheaves should be cleaned and painted. The moderate rehabilitation includes the replacement of all rollers and sheaves. The major rehabilitation includes the replacement of all operating drums, sheaves and rollers with improved pitch diameter equivalents. It is recommended that the pitch diameter of these components be increased to at least 45”for a 1” diameter operating rope. If the arc of contact between a deflector sheave and the rope is less than 45 degrees a pitch diameter of 26” for a 1” diameter operating rope is acceptable.

4.3 Counterweight Assemblies:

The counterweight assemblies on the main movable span do not shown significant deterioration of condition from the previous report. The counterweight assembly components are a mix of the original installation and new components installed through various rehabilitation contracts over the years. As noted in previous inspections the counterweight equalizing assemblies creak during the operation of the movable span. The noise was most notable on the south counterweight equalizing assemblies, but the maintenance staff noted that this side was due for lubrication which had not been performed yet.

A limited investigation of the rope tensions was performed at return site visit in October. The American Bridge method of checking rope tensions was used on a random selection of ropes in all four corners of the movable span. Six ropes were randomly selected in each corner of the movable span. The resulting calculated rope tensions show that the rope tensions do vary significantly for the ropes selected. The worst case variation occurred in the northwest corner of the bridge where the highest varying rope was 13.29% above the mean tension and the lowest varying rope was 17.15% below the mean tension. The Southwest corner exhibited the least amount of variation where the highest varying rope was 9.36% above the mean tension and the lowest varying rope was 6.26% below the mean tensions. The variation of the rope tensions does not meet the industry acceptance criteria of 2 ½ % variation between individual ropes, as interpreted from AASHTO (1988 ed. 4.1.6). The summarized rope tensions for each corner can be found in appendix C.

4.3.1 Counterweight Sheaves and Bearings:

There are 4 sets of counterweight sheaves and bearings for the main movable span, one in each corner of the span. The counterweight sheaves and bearings carry the entire weight of the movable span, counterweight and counterweight ropes. The counterweight sheaves are steel castings with machined rope grooves. The counterweight sheaves are supported by plain style trunnion bearings with bronze bushings.

During operation of the movable span it was noted that there was an intermittent banging noise coming from the Northwest counterweight sheave. Such noise is usually associated with a difference in the tension of the counterweight ropes in that corner of the span. Further discussion on the tension of the counterweight ropes will be presented later in section 4.3.2. The counterweight sheave rope grooves were checked for wear with the appropriate sized gauge, and no significant wear was found. The report of significant wear in the first groove the southwest sheave could not be confirmed. The crack in the outboard web of the NE sheave was confirmed and showed no additional growth since the previous inspection (See photo M-13). The condition of the trunnion shaft journal and transition radius could not be assessed at the time of the scoping inspection. The trunnion bearings were not disassembled. The trunnion bearings were visually inspected and clearance measurements were taken. A table of bearing clearance field measurements can be found in appendix C. Previous inspections noted scoring and wear of the trunnion shaft journals when the trunnion bearings were disassembled. It is assumed that these conditions have not significantly changed since the last inspection. The only other deficiencies the counterweight sheaves, bearings, and trunnion shafts are minor maintenance deficiencies related to paint failure and surface corrosion.

Several options are possible for the rehabilitation of the counterweight sheave deficiencies. The least costly approach would be to perform no action on the counterweight rope grooves and allow the grooves to wear further. This approach will most likely not affect the service life of the sheave itself, but will create accelerated wear on the counterweight ropes. This wear on the counterweight ropes would be regardless of whether the existing ropes or new ropes are installed. The moderate rehabilitation alternative would address any of the deficiencies previously noted with trunnion shaft journal. The alternative would skim cut the counterweight trunnion shafts to as slightly smaller diameter. The machining would remove potential surface deficiencies and reduce the acceleration of the wear between the trunnion shaft and the trunnion bushings. This option is only available if an analysis of the trunnion shaft, bearings and sheave show that the reduced shaft section can accommodate the current stresses and still meet the requirement of current movable bridge code. This analysis is outside of the scope of this report and should be performed prior to consideration of the moderate rehabilitation alternative. Finally, the major rehabilitation alternative would be a complete replacement of the counterweight sheave. The major rehabilitation alternative is the most costly option, but it also guarantees the desired design life for the counterweight sheave.

Regardless of the work performed on the counterweight sheave in the first two options, the trunnion bearings should be cleaned and painted with an inspection of the trunnion shaft journal. If the counterweight sheave is to be replaced, then the bushings in the trunnion bearings should also be replaced.

4.3.2 Counterweight Ropes and Connections:

The counterweight ropes are 1 5/8" diameter Improved Plow Steel (IPS) 6x25 filler wire construction with a fiber core. Each rope has an open spelter socket at both ends to connect to the counterweight and the movable span. The counterweight ropes are connected at the movable span via threaded bar rope take-up mounted in steel structural supports. The counterweight ropes are connected to the counterweight via an equalizing device consisting of a collection of bars and pins that bring the connection of the counterweight ropes down to a single pin connection at the counterweight. During inspection of the counterweight ropes it was noted that there were new ropes installed in both the northwest and southwest corners of the movable span. It was later confirmed that the ropes were replaced as part of a rehabilitation contract in May of 2008.

During a previous inspection it was noted that several broken wires were found at the tangent point of the counterweight ropes on the span side of the counterweight sheaves in the northwest corner. The current inspection did not operate the movable span to confirm this condition and could not verify the condition of the broken wires. Lay length and rope diameter measurements were taken from on top of the movable span truss at the span connection of the counterweight ropes, a table of values can be found in appendix C. In general the rope lay length and diameter measurements were found to be acceptable. The wire ropes were found to be in good condition other than the broken wires noted in the previous report.

The counterweight rope connection at the movable span was visually inspected and only minor deficiencies related to paint and surface corrosion were noted.

The counterweight connection at the counterweight and the equalizing device was visually inspected. Where possible, the clearance at the equalizing device pins was measured and a table of measurements is presented in appendix C. The worn clearances between the pins and bars of the counterweight equalizing device have not increased since the previous inspection (See photo M-14).

As noted previously, the counterweight rope unequal tensions are the most serious problem with the counterweight ropes and will affect the remaining effective service life of the counterweight ropes. The unequal rope tensions also suggest along with the creaking noise emitted from the equalizing device on the counterweight, that the equalizing device is not working properly and that some of the plate and pin connections may be seized.

The visual inspection of the ropes did not identify any serious surface or construction deficiencies with the counterweight ropes. Maintenance and lubrication of the counterweight ropes appeared to be adequate.

Three different levels of rehabilitation work are possible for the counterweight ropes and connections. Minor rehabilitation should consist of cleaning and painting all counterweight rope connection and adjusting the tension of counterweight ropes. The moderate rehabilitation includes the replacement of all remaining counterweight ropes along with the cleaning and painting of the counterweight rope connections. Major rehabilitation of the counterweight ropes includes the replacement of all counterweight ropes and connections to the movable span and counterweight. The equalizing device on used as the counterweight connection would be replaced with a fixed type connection for each counterweight rope.

4.3.3 Counterweight Guides:

The counterweight guides are located laterally on either side of the counterweight and engage rails along the inside of the tower legs. The guides were not accessible to be closely inspected. Given the condition of the guide rail and the fact that the guides did not seem to be riding unevenly, the counterweight guides appear to be in good condition. Lubrication of the counterweight guides and rails was found to be acceptable. It is recommended that the counterweight guides be cleaned and painted in all rehabilitation alternatives.

4.4 Live Load Bearings And Span Guides

4.4.1 Live Load Bearings:

There are two live load bearings located at the corners of the span, on either end of the span, at the rest pier. They consist of an upper bearing half on the movable span and a lower bearing half mounted on the rest pier. The North live load bearings are expansion bearing and the South live load bearings are fixed bearings. All of the live load bearings were found to have only minor deficiencies, with no visible, significant wear (See photo M-15). Regardless of the rehabilitation alternative, it is suggested that all of the live load bearings be cleaned, painted and the lower half of the live load bearings be reset on the pier with new anchor bolts.

4.4.2 Span Guides:

There are two sets of span guides located on either end of the span. The lower set is located at the bottom chord of the truss while the upper set is located at the top chord of the truss. All span guides engage a rail that runs along the inside of the tower leg. All of the span guides were found to have only minor deficiencies (See photo M-16). During inspection it was noted that the bolts for the wear plate on the Northwest lower span guide were loose and should be tightened to prevent the wear pad from falling off (See photo M-17). Regardless of the rehabilitation alternative, it is suggested that the span guides be cleaned, painted, and the wearing plates be replaced same-in-kind.

4.5 Span Balance:

The span balance of the main movable span was not directly measured during the operation of the span. Given information from the wear on the rack and pinion teeth, and amperage readings recorded during operating it is assumed that the main movable span is span heavy through the first half of the lift. After the first half of the lift, enough of the counterweight ropes have shifted over to the counterweight side to create a counterweight heavy condition. The Memorial Bridge does not have any provisions to offset the weight of the counterweight ropes as the span travels from fully open to fully closed. No specific work is recommended for the span balance. Span balance adjustment may be necessary if there is significant structural work on the movable span that may change the overall weight of the movable span.

5 RECOMMENDATIONS

5.1 Minimum Rehabilitation:

1. Operating Machinery
    a. Rehabilitate Primary and Auxiliary Reducers: new shaft seals, breather and, paint housing.
    b. Replace the LPG back-up engine transmission and clutch.
    c. Clean and paint bearings, shafts, and couplings.
    d. Adjust bearing liners for proper bearing clearance.
    e. Replace drum gears and pinion shafts.
    f. Clean operating drum rope grooves.
    g. Mounting of full open and over-travel limit switches.

2. Operating Ropes
    a. Tension operating ropes.
    b. Clean and lubricate operating ropes.
    c. Monitor condition of Northeast uphaul rope.
    d. Replace operating rope rollers.
    e. Clean and paint operating rope deflector and intermediate sheaves.

3. Counterweight Assemblies
    a. Clean, paint and lubricate counterweight rope equalizer.
    b. Check and adjust counterweight rope tensions by working to free the equalizing device.
    c. Clean and paint counterweight guides.
    d. Clean and paint counterweight sheave trunnion shaft bearings.
    e. Clean counterweight sheave rope grooves.

4. Live load bearings and Span Guides
    a. Clean, paint and replace anchor bolts for live load bearings.
    b. Clean, paint and replace wear plates on span guides.

5. Span Balance
    a. No expected span balance work.

5.2 Moderate Rehabilitation:

1. Operating Machinery
    a. Mount and align new motors with new grid type motor couplings.
    b. Mount and align new thruster brakes.
    c. Rehabilitate Primary and Auxiliary Reducers: new shaft seals, breather and, paint housing.

d. Replace the LPG back-up engine transmission and clutch.
e. Clean and paint bearings, shafts, and couplings.
f. Adjust bearing liners for proper bearing clearance.
g. Replace drum gears and pinion shafts.
h. Clean operating drum rope grooves.

2. Operating Ropes

a. Replace damaged Northeast uphaul rope
b. Tension operating ropes.
c. Clean and lubricate operating ropes.
d. Replace operating rope rollers, deflector sheaves, and intermediate sheaves.

3. Counterweight Assemblies

a. Clean, paint and lubricate counterweight rope equalizer.
b. Replace all remaining counterweight ropes.
c. Clean and paint counterweight guides.
d. Machine counterweight trunnion shaft journals and replace trunnion bearing bushings.
e. Clean counterweight sheave rope grooves.

4. Live load bearings and Span Guides

a. Replace anchor bolts for live load bearings.
b. Replace wear plates on span guides.

5. Span Balance

a. No expected span balance work.

5.3 Major Rehabilitation:

1. Operating Machinery

a. Mount and align new motors with new grid type motor couplings.
b. Mount and align new thruster brakes.
c. Mount and align new primary reducer.
d. Install new bearings, shafts and couplings.
e. Install new operating rope drums with new pitch diameter, including rack and pinion gears.

2. Operating Ropes

a. Install new operating ropes.
b. Replace operating rope rollers, deflector sheaves, and intermediate sheaves. New rollers and sheaves to have new pitch diameter.

3. Counterweight Assemblies

    a. Replace all counterweight ropes.
    b. Replace counterweight rope connections, including replacement of the equalizing devices with a fixed connection for each counterweight rope.
    c. Clean and paint counterweight guides.
    d. Replace counterweight sheaves, trunnion shafts, and trunnion shaft bearings.

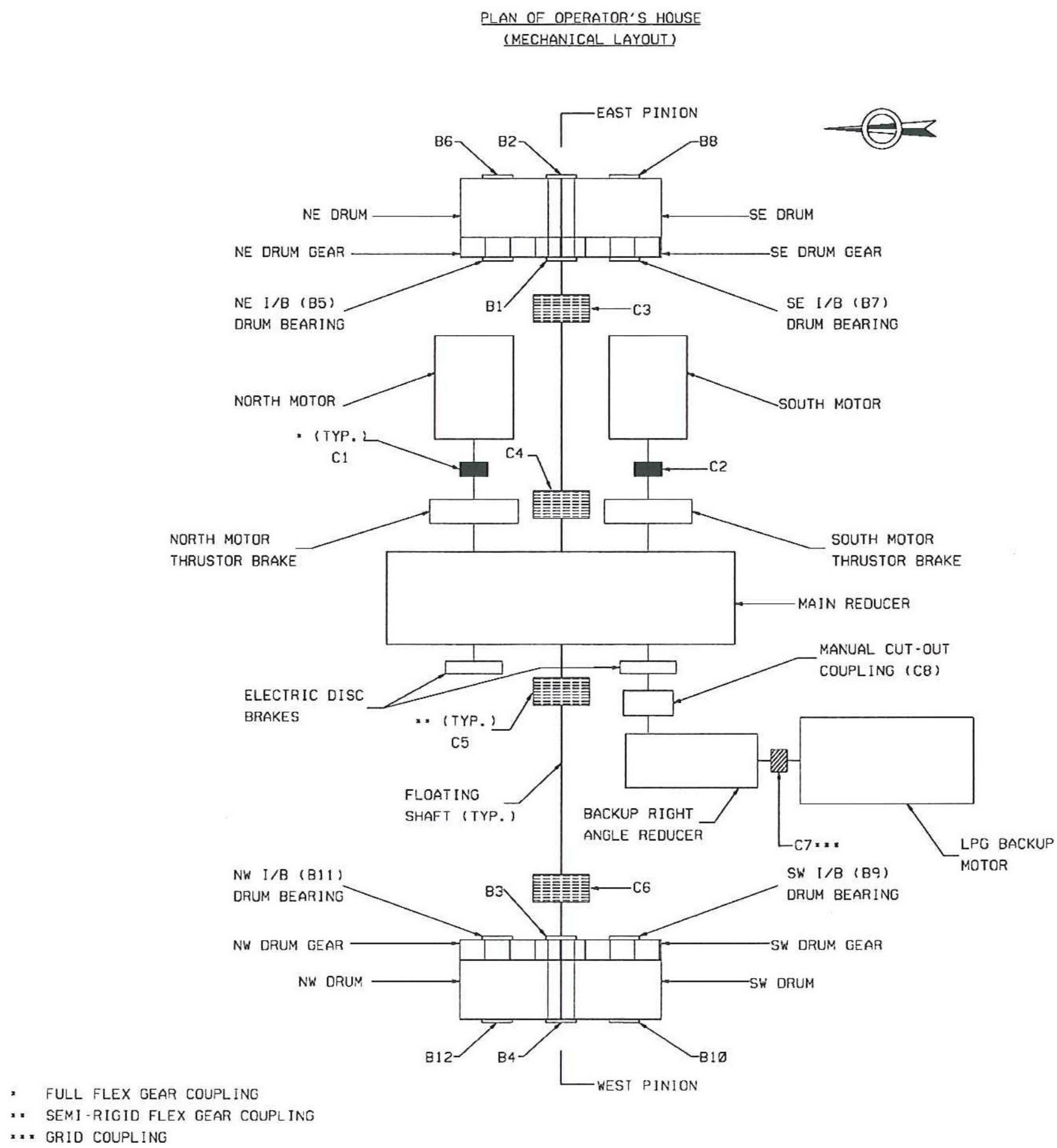
4. Live load bearings and Span Guides

    a. Replace live load bearings.
    b. Replace span guides.

5. Span Balance

    a. Adjust span balance of counterweight with respect to new span.

Appendix A
Machinery Diagram


PLAN OF OPERATOR'S HOUSE
(MECHANICAL LAYOUT)
EAST PINION
B6
B2
B8
NE DRUM
SE DRUM
NE DRUM GEAR
SE DRUM GEAR
NE I/B (B5)
DRUM BEARING
B1
C3
SE I/B (B7)
DRUM BEARING
NORTH MOTOR
SOUTH MOTOR
* (TYP.)
C1
C4
C2
NORTH MOTOR
THRUSTOR BRAKE
SOUTH MOTOR
THRUSTOR BRAKE
MAIN REDUCER
MANUAL CUT-OUT
COUPLING (C8)
ELECTRIC DISC
BRAKES
** (TYP.)
C5
FLOATING
SHAFT (TYP.)
BACKUP RIGHT
ANGLE REDUCER
C7***
LPG BACKUP
MOTOR
NW I/B (B11)
DRUM BEARING
B3
C6
SW I/B (B9)
DRUM BEARING
NW DRUM GEAR
SW DRUM GEAR
NW DRUM
SW DRUM
B12
B4
B10
WEST PINION
* FULL FLEX GEAR COUPLING
** SEMI-RIGID FLEX GEAR COUPLING
*** GRID COUPLING

Appendix B
Photos

Photo M-1: The thruster brakes are old style GE brakes and are no longer commercially available products.

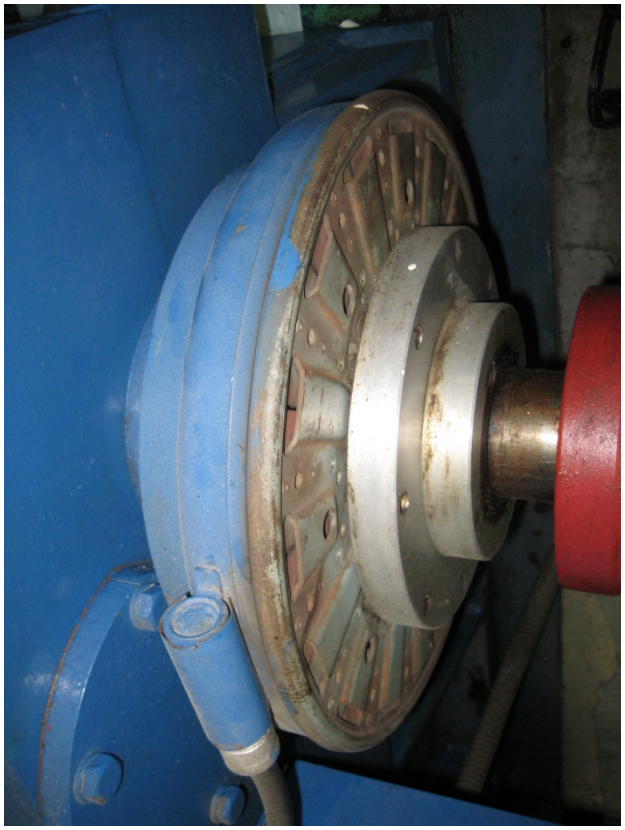


Photo M-2: The electro-magnetic disc brakes are old and are no longer commercially available products.

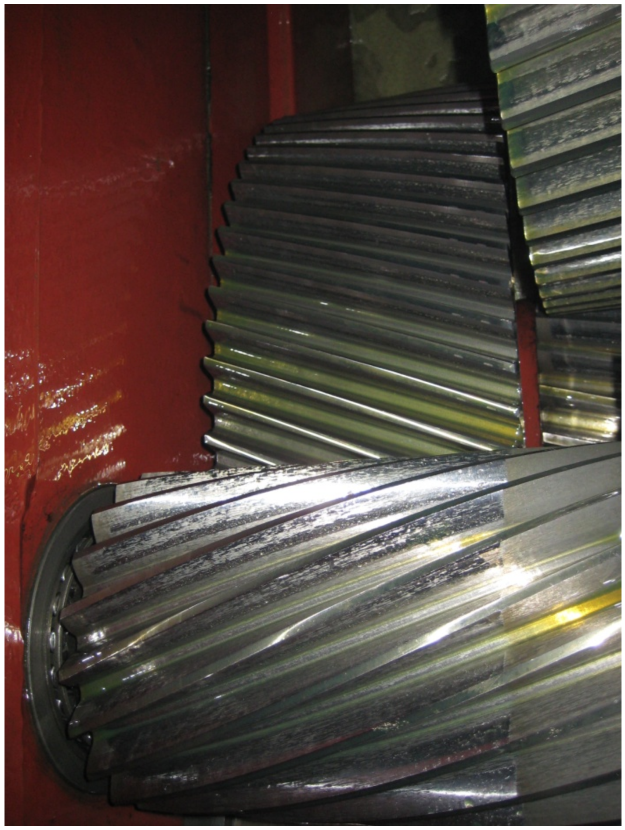


Photo M-3: Both reducers suffer from moisture intrusion into the interior of the housing.

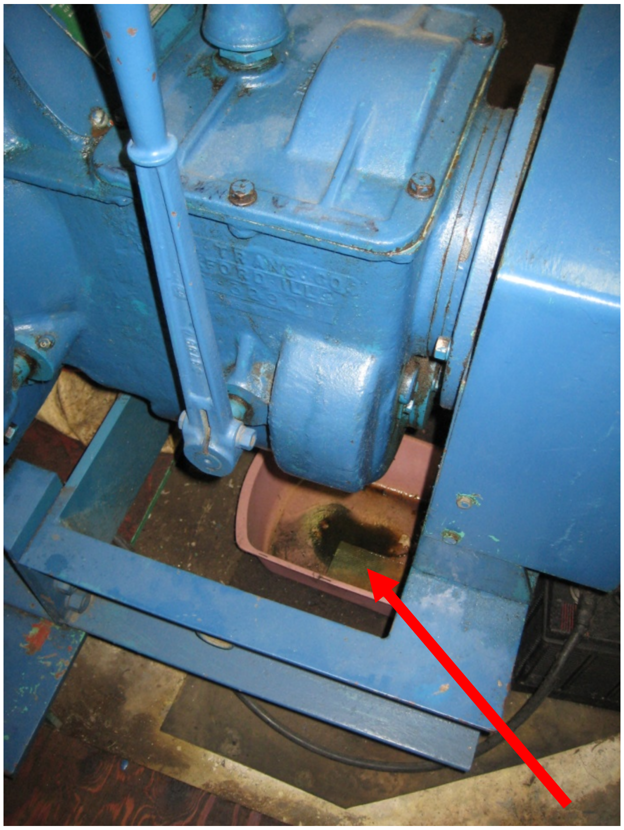


Photo M-4: The LPG back-up motor transmission leaks oil.

Photo M-5: Most shafts and couplings have paint related deficiencies.

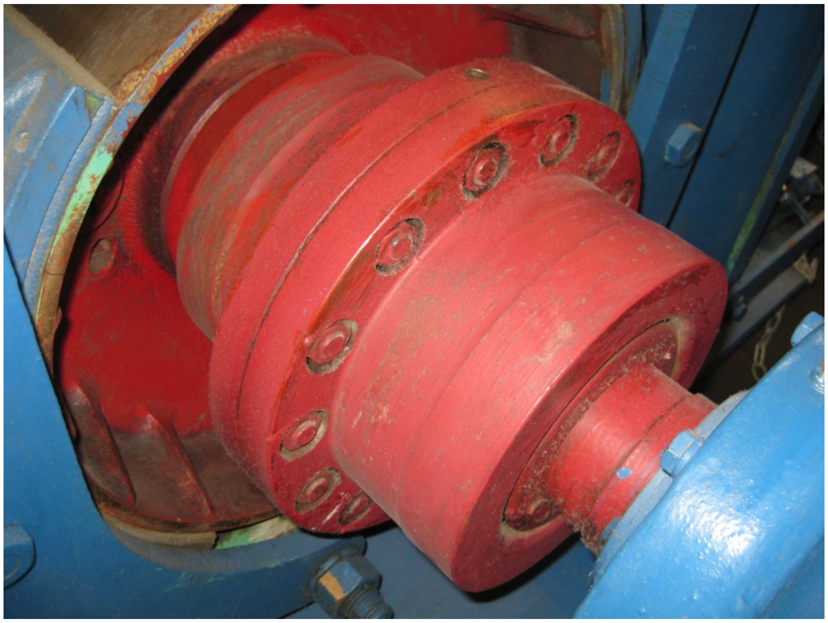


Photo M-6: It should be noted that the motor couplings C1 and C2 are gear type couplings.

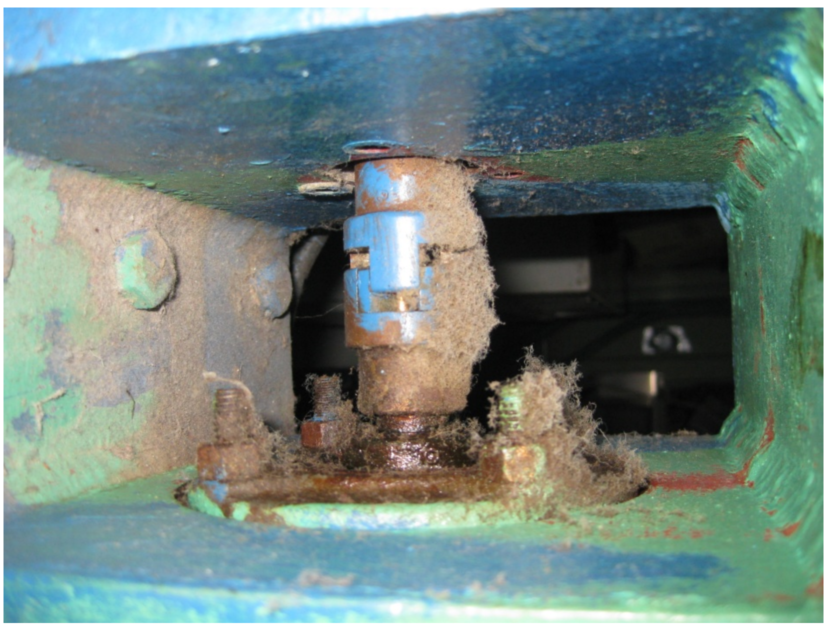


Photo M-7: Coupling C-11 may have a loose hub which does not fully engage the center spider of the coupling.

Photo M-8: The machinery bearings have minor maintenance related deficiencies and in most cases have wear beyond the standards for clearance.

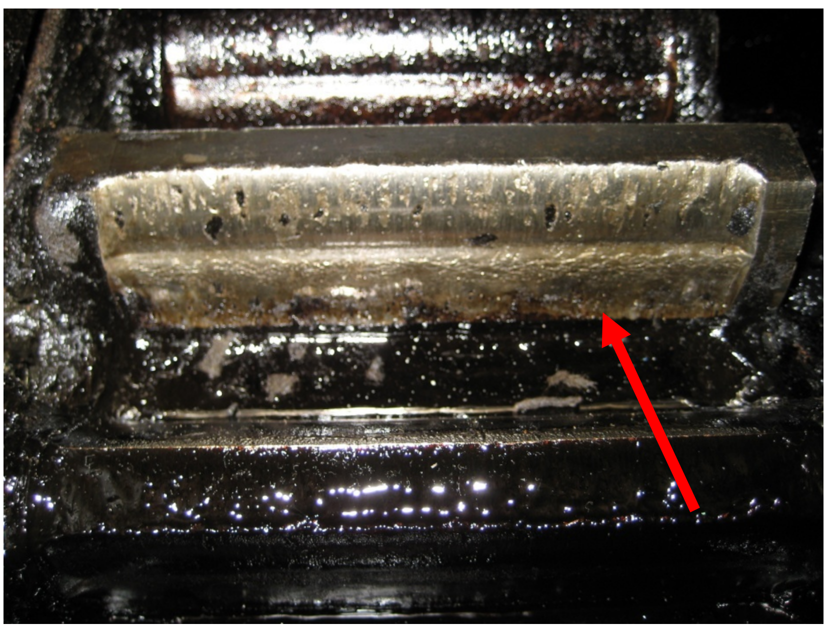


Photo M-9: The gear teeth show significant wear in all the open gear sets of the movable span, Southeast drum gear raise face shown.

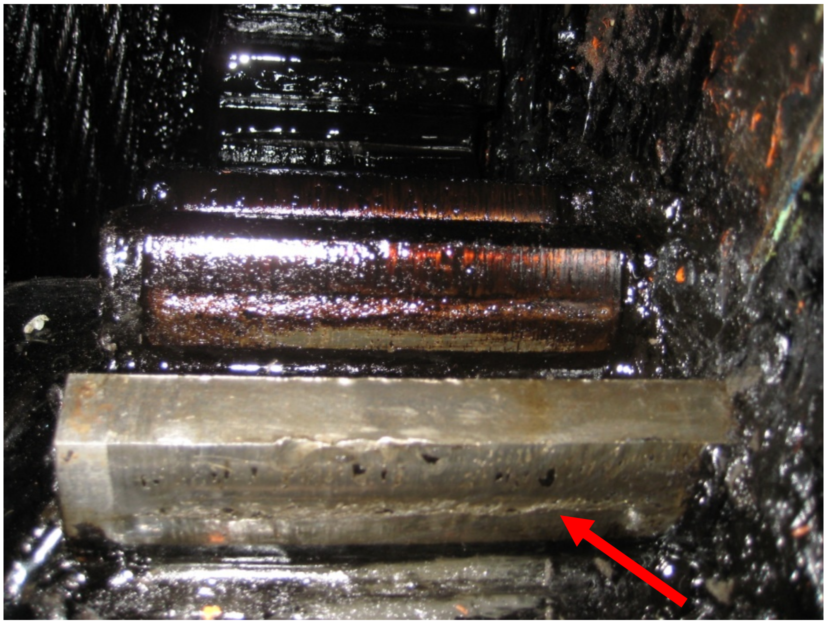


Photo M-10: The gear teeth show significant wear in all the open gear sets of the movable span, West pinion gear raise face shown.

Photo M-11: There is a kink in the East, northeast uphaul rope at about the just before the double rollers when the span was approximately 1/3 open position. The kink in the rope was visible when the load on the rope was relieved and the rope went slack.

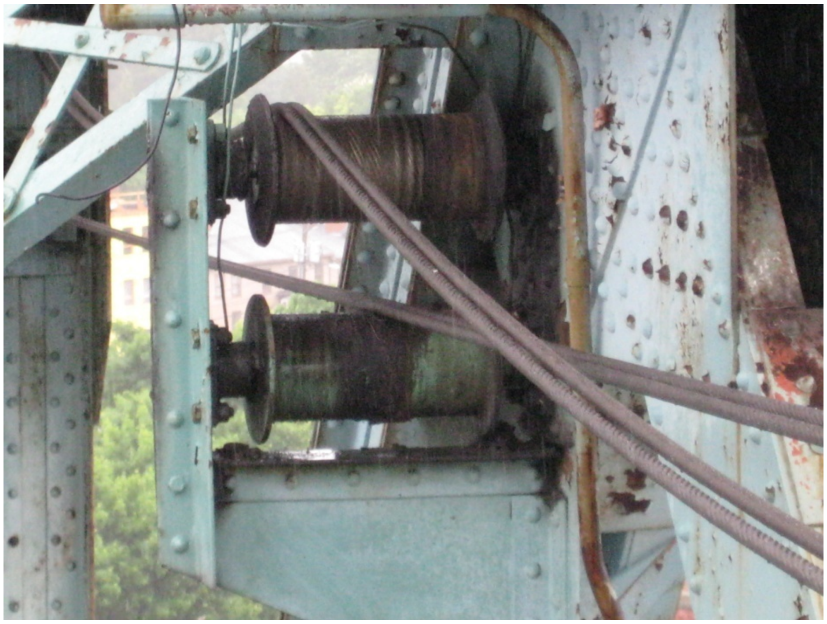


Photo M-12: These rollers show evidence of groves and scoring over the outer diameter of the rollers caused by contact with the operating ropes.

Photo M-13: The crack in the outboard web of the NE sheave was confirmed and showed no additional growth since the previous inspection.

Photo M-14: The worn clearance between the pins and bars of the counterweight equalizing device have not increased since the previous inspection.

Photo M-15: All of the live load bearings were found to have only minor deficiencies, with no visible, significant wear.

Photo M-16: All of the span guides were found to have only minor deficiencies such as paint failure and corrosion.

Photo M-17: During inspection it was noted that the bolts for the wear plate on the Northwest lower span guide were loose.

Appendix C
Field Measurements

Bearing Clearances:

Bearing Mark, Location Description	Previous Clearance	Measured Clearance	Original Fit Clearance
Northeast East Trunnion Bearing	N/A	0.035" +	0.015"
Northeast West Trunnion Bearing	N/A	0.055" +	0.015"
Northwest East Trunnion Bearing	N/A	0.048" +	0.015"
Northwest West Trunnion Bearing	N/A	0.035" +	0.015"
Southeast East Trunnion Bearing	N/A	0.023" +	0.015"
Southeast West Trunnion Bearing	N/A	0.007" +	0.015"
Southwest East Trunnion Bearing	N/A	0.035" +	0.015"
Southwest West Trunnion Bearing	N/A	0.037" +	0.015"
Bearing B1, East inboard Pinion	0.022"	No Measure	0.010"
Bearing B2, East outboard pinion	N/A	0.008"	0.010"
Bearing B3, West inboard pinion	N/A	0.025" *	0.010"
Bearing B4 West outboard pinion	N/A	0.006"	0.010"

+ Bearing clearance noted is between the trunnion journal and cap, not a running fit.

*Requires immediate attention.

Bearing Clearances (Cont.):

Bearing Mark, Location Description	Previous Clearance	Measured Clearance	Original Fit Clearance
Bearing B5, NE inboard drum	0.017”	0.020”	0.010”
Bearing B6, NE outboard drum	N/A	No Measure	0.010”
Bearing B7, SE Inboard drum	0.016”	0.017”	0.010”
Bearing B8, SE outboard drum	N/A	No Measure	0.010”
Bearing B9, SW inboard drum	0.022”	0.024” *	0.010”
Bearing B10, SW outboard drum	N/A	No Measure	0.010”
Bearing B11, NW Inboard drum	N/A	0.025” *	0.010”
Bearing B12, NW outboard drum	N/A	No Measure	0.010”

+ Bearing clearance noted is between the trunnion journal and cap, not a running fit.

*Requires immediate attention.

Gear Tooth Measurements:

<table>
<tr><th colspan="5">Tooth Thickness

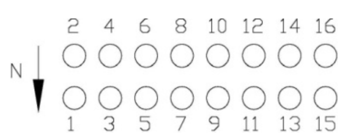
Pinion Gears measured at addendum = 0.662”

Drum Gears measured at addendum = 0.645”</th></tr>
<tr><th></th><th colspan="2">Chordal</th><th colspan="2">Backlash</th></tr>
<tr><th>Gear Mark</th><th>Previous</th><th>Measured</th><th>Previous</th><th>Measured</th></tr>
<tr><td rowspan="2">Drum Gear Northeast</td><td rowspan="2">1.205”</td><td rowspan="2">T- 1.198”
M- 1.205”
H- 1.205”</td><td rowspan="3">0.110”</td><td rowspan="3">0.1137”</td></tr>
<tr></tr>
<tr><td rowspan="2">East Pinion</td><td rowspan="2">1.205”</td><td rowspan="2">T- 1.093”
M- 1.092”
H-1.095”</td></tr>
<tr><td rowspan="3">0.047”</td><td rowspan="3">0.114”</td></tr>
<tr><td rowspan="2">Drum Gear Southeast</td><td rowspan="2">1.205”</td><td rowspan="2">T- 1.200”
M- 1.204”
H-1.203”</td></tr>
<tr></tr>
<tr><td rowspan="2">Drum Gear Northwest</td><td rowspan="2">1.205”</td><td rowspan="2">T- 1.120”
M- 1.120”
H- 1.190”</td><td rowspan="3">0.069”</td><td rowspan="3">0.065”</td></tr>
<tr></tr>
<tr><td rowspan="2">West Pinion</td><td rowspan="2">1.205”</td><td rowspan="2">T- 1.220”
M- 1.190”
H-1.200”</td></tr>
<tr><td rowspan="3">0.047”</td><td rowspan="3">0.098”</td></tr>
<tr><td rowspan="2">Drum Gear Southwest</td><td rowspan="2">1.205”</td><td rowspan="2">T- 1.184”
M- 1.180”
H-1.184”</td></tr>
<tr></tr>
</table>

Counterweight Rope Measurements:

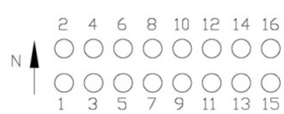
Southwest Corner					
Rope Number	Average Period (seconds)	Tension (lb)	Deviation	Lay Length	Rope Diameter
1	8.18	37157.41	8.76%	No Measure	No Measure
2	No Measure	No Measure	No Measure	10.800”	1.635”/1.638”
3	No Measure	No Measure	No Measure	10.569”	1.628”/1.639”
4	8.70	32869.96	-3.14%	No Measure	No Measure
5	8.30	36064.25	6.00%	No Measure	No Measure
6	No Measure	No Measure	No Measure	11.250”	1.632”/1.635”
7	No Measure	No Measure	No Measure	10.700”	1.635”/1.630”
8	No Measure	No Measure	No Measure	No Measure	No Measure
9	No Measure	No Measure	No Measure	No Measure	No Measure
10	No Measure	No Measure	No Measure	10.644”	1.631”/1.644”
11	8.38	35417.81	4.28%	10.700”	1.632”/1.634”
12	8.57	33818.51	-0.24%	No Measure	No Measure
13	No Measure	No Measure	No Measure	No Measure	No Measure
14	No Measure	No Measure	No Measure	10.495”	1.649”/1.647”
15	No Measure	No Measure	No Measure	10.850”	1.631”/1.634”
16	9.41	28075.44	-20.75%	No Measure	No Measure
Average Tension (lb)		33900.56			

Rope Layout at Lifting Girder:



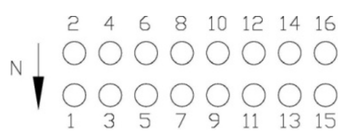
Northwest Corner					
Rope Number	Average Period (seconds)	Tension (lb)	Deviation	Lay Length	Rope Diameter
1	8.42	34896.38	6.07%	10.231”	1.649”/1.649”
2	No Measure	No Measure	No Measure	No Measure	
3	No Measure	No Measure	No Measure	10.744”	1.651”/1.651”
4	9.40	27979.56	-17.15%	10.770”	1.627”/1.635”
5	No Measure	No Measure	No Measure	No Measure	No Measure
6	No Measure	No Measure	No Measure	No Measure	No Measure
7	No Measure	No Measure	No Measure	No Measure	No Measure
8	No Measure	No Measure	No Measure	10.762”	1.638”/1.645”
9	8.53	34015.74	3.64%	10.824”	1.666”/1.758”
10	8.56	33817.70	3.07%	No Measure	No Measure
11	No Measure	No Measure	No Measure	No Measure	No Measure
12	No Measure	No Measure	No Measure	10.683”	1.634”/1.632”
13	No Measure	No Measure	No Measure	10.385”	1.651”/1.664”
14	No Measure	No Measure	No Measure	No Measure	No Measure
15	8.11	37801.88	13.29%	No Measure	No Measure
16	9.37	28162.34	-16.39%	10.700”	1.624”/1.629”
Average Tension (lb)		32778.93			

Rope Layout at Lifting Girder:



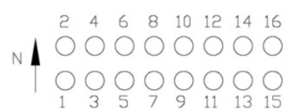
Southeast Corner					
Rope Number	Average Period (seconds)	Tension (lb)	Deviation	Lay Length	Rope Diameter
1	8.09	37971.25	9.36%	No Measure	No Measure
2	No Measure	No Measure	No Measure	No Measure	No Measure
3	8.77	32389.42	-6.26%	No Measure	No Measure
4	No Measure	No Measure	No Measure	No Measure	No Measure
5	No Measure	No Measure	No Measure	No Measure	No Measure
6	8.59	33892.58	-1.55%	No Measure	No Measure
7	No Measure	No Measure	No Measure	No Measure	No Measure
8	No Measure	No Measure	No Measure	No Measure	No Measure
9	No Measure	No Measure	No Measure	No Measure	No Measure
10	8.46	34655.74	0.69%	No Measure	No Measure
11	8.51	34354.10	-0.18%	No Measure	No Measure
12	No Measure	No Measure	No Measure	No Measure	No Measure
13	No Measure	No Measure	No Measure	10.800”	1.630”/1.631”
14	No Measure	No Measure	No Measure	11.113”	1.636”/1.640”
15	No Measure	No Measure	No Measure	10.985”	1.628”1.642”
16	8.64	33236.05	-3.55%	10.770”	1.635”/1.632”
Average Tension (lb)		34416.52			

Rope Layout at Lifting Girder:



Northeast Corner					
Rope Number	Average Period (seconds)	Tension (lb)	Deviation	Lay Length	Rope Diameter
1	No Measure	No Measure	No Measure	10.760”	1.626”/1.634”
2	9.18	29478.99	-11.11%	10.670”	1.630”/1.640”
3	8.59	33627.89	2.60%	10.911”	1.634”/1.640”
4	9.16	29664.36	-10.42%	10.687”	1.630”/1.651”
5	No Measure	No Measure	No Measure	No Measure	No Measure
6	No Measure	No Measure	No Measure	No Measure	No Measure
7	No Measure	No Measure	No Measure	No Measure	No Measure
8	8.67	33.35.57	0.85%	No Measure	No Measure
9	8.31	35935.17	8.85%	No Measure	No Measure
10	No Measure	No Measure	No Measure	No Measure	No Measure
11	No Measure	No Measure	No Measure	No Measure	No Measure
12	No Measure	No Measure	No Measure	No Measure	No Measure
13	No Measure	No Measure	No Measure	No Measure	No Measure
14	No Measure	No Measure	No Measure	No Measure	No Measure
15	8.46	34786.97	5.84%	No Measure	No Measure
16	No Measure	No Measure	No Measure	No Measure	No Measure
Average Tension (lb)		32754.82			

Rope Layout at Lifting Girder:



Operating Rope Measurements:

Rope Location	Lay Length	Rope Diameter
Northwest	No Measure	No Measure
Southwest		
Raise East	12.830”	1.018”
Raise West	12.660”	1.025”
Lower East	13.087”	1.012”
Lower West	13.085”	1.025”
Northeast	No Measure	No Measure
Southeast		
Raise East	12.780”	1.031”
Raise West	12.700”	1.014”
Lower East	12.800”	1.028”
Lower West	13.085”	1.024”

Counterweight Rope Sheave Groove Measurements:

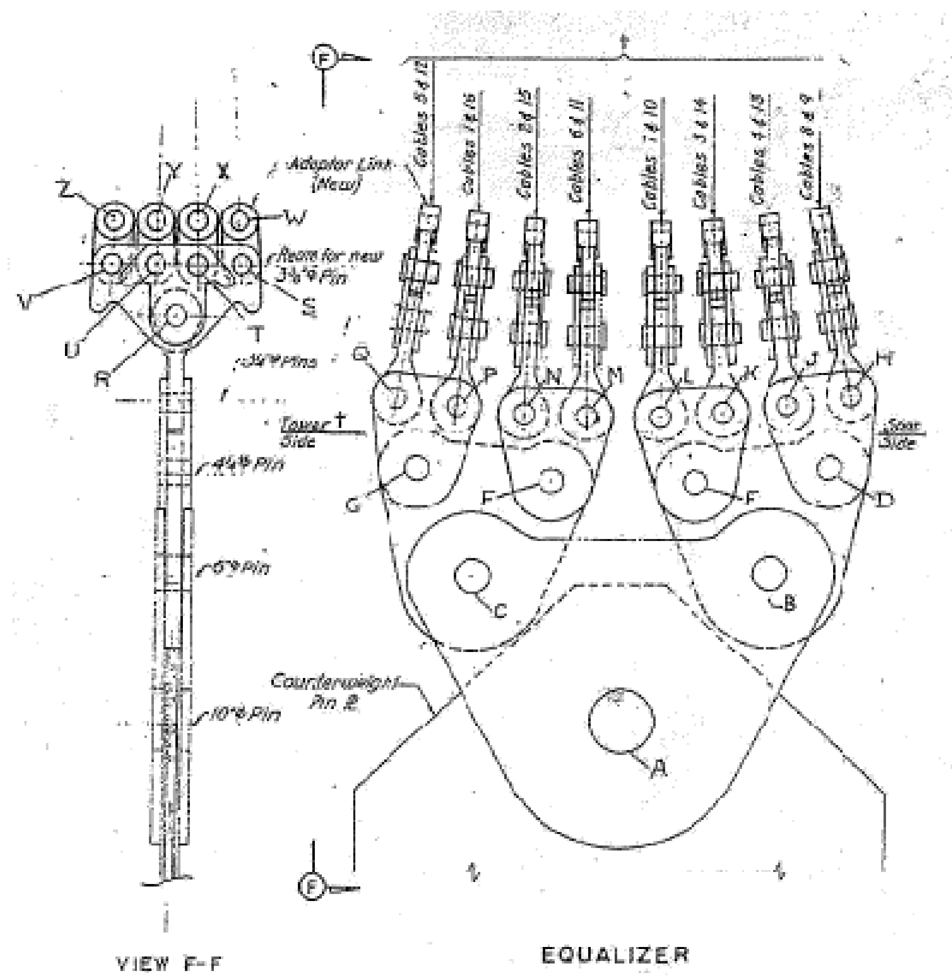
All Grooves Measured with 1 5/8” + 3/64” Guage				
Groove Number	Northeast	Northwest	Southeast	Southwest
1	Worn	Worn	Worn	Worn
2	Worn	Worn	Worn	Worn
3	Worn	Worn	Worn	Worn
4	Worn	Worn	Worn	Worn
5	Worn	Worn	Worn	Worn
6	Worn	Worn	Worn	Worn
7	Worn	Worn	Worn	Worn
8	Worn	Worn	Worn	Worn
9	Worn	Worn	Worn	Worn
10	Worn	Worn	Worn	Worn
11	Worn	Worn	Worn	Worn
12	Worn	Worn	Worn	Worn
13	Worn	Worn	Worn	Worn
14	Worn	Worn	Worn	Worn
15	Worn	Worn	Worn	Worn
16	Worn	Worn	Worn	Worn

Counterweight Rope Equalizer Pin Clearance Measurements:

*Measurements of pin clearance were only possible at the tertiary level pins				
Groove Number	Northeast	Northwest	Southeast	Southwest
D	3/4“	3/16 “	3/8”	3/8”
E	0.069”	No Measure	No Measure	No Measure
F	No Measure	0.085”	0”	0”
G	9/16”	1/2“	0.135”	0”

*

Pin Naming Convention:



Appendix D
Field Inspection Sheets

Memorial Bridge Span Features

Performed by: 5/28/09

Date : KPK

Span Guides

Shaft Mark, Location Description	Exterior Conditon, Bolts and Housing	Remarks
Northeast upper exp	- Some corr on fastners + steel - No noticeable wear.	
Northeast lower exp	Good clearance, surface corrosion	
Northwest ~~lower~~ upper exp	- Some corr on fastners & steel - No noticeable wear	
Northwest lower exp	Surf corrosion loose Bolts	

Memorial Bridge Span Features

Performed by:

Date :

Span Guides

Shaft Mark, Location Description	Exterior Conditon, Bolts and Housing	Remarks
Southeast upper	cannot inspect	
Southeast lower	slight corrosion Clearance OK	
Southwest ~~lower~~ upper	Some corr on fastners No noticable wear	
Southwest lower	slight corrosion Clearance OK	

Memorial Bridge Span Features

Performed by: MPM

Date : 5/08/09

Live Load Bearings

Shaft Mark, Location Description	Exterior Conditon, Bolts and Housing	Remarks
Northeast	Paint failure surface corrosion	
Northwest South	Paint Failure and corrosion	
Southeast	slight pumping under live load Paint failure corrosion not seated	
Southwest North	Paint failure surface corrosion	

Project:	Computed:	Date:
Subject:	Checked:	Date:
Task:	Page:	of:
Job #:	No:	

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Counterweight Ropes
Memorial Bridge

16 14 12 10 8 6 4 2
15 13 11 9 7 5 3 1

N

Northwest

Rope #	Lay Length	Dia	Condition
2	10,231	1.649 1.649	new Poor lube
3	10,770	1.627 1.635	
4	10,744	1.651 1.651	
7	10,762	1.638 1.645	
10	10,824	1.666 1.758	← * dirty
11	10,683	1.634 1.632	
14	10,385	1.651 1.664	new
15	10,100	1.629 1.629	

Project:	Computed:	Date:
Subject:	Checked:	Date:
Task:	Page:	of:
Job #:	No:	

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Counterweight Ropes
Memorial Bridge

4 0 0 2
0 0
3 1

↓N

Northeast

Rope #	Lay length	Dia.	Condition
		1.630	Poor Wire
1	10.670	1.640	
2	10.760	1.626 1.634	
3	10.911	1.634 1.640	
4	10.687	1.630 1.651	

Project:	Computed:	Date:
Subject:	Checked:	Date:
Task:	Page:	of:
Job #:	No:	

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16 14 12 10 8 6 4 2

15 13 11 9 7 5 3 1

Memorial Bridge N ↓

Counterweight Ropes

South West Corner

Rope	Lay	Dia	Condition
1	10.850	1.631 1.634	Poor Lube
4	10.495	1.649 1.647	new Rope
5	10.700	1.632 1.634	
9	10.700	1.635 1.630	
12	11.250	1.632 1.635	
13	10.569	1.628 1.639	
16	10.800	1.635 1.638	
8	10.644	1.631 1.644	

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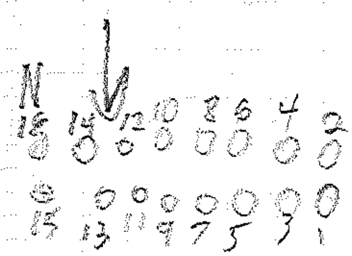
Project: Memorial Bridge		Computed:	Date:
Subject: SE Counterweight Ropes		Checked:	Date:
Task:		Page:	of:
Job #:		No:	

	Lay	Dia	~~comment~~ condition
1	10.985	1.628 1.642	poor lube
2	10.770	1.635 1.632	
3	10.800	1.630 1.631	
4	11,113	1.636 1.640	



Counterweight System

Performed by: KC

Date : 5/27

*See additional measurements

replacements and hallo poor lube

Counterweight Ropes

Shaft Mark, Location Description	Exterior Conditon	Rope Diameter Measurements	Lay Length Measure	Condition of Wire Rope				
				Broken Wires	Abrasion of Wires	Pitting of Wires	Peening of Wires	Crown Wear
Northwest	2 Ropes appear to be: furthest east Rope poor lubrication 3 from west poor lub							
Southwest	lubrication good.							

Notes:

Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:		Page:	of:
Job #:		No:	

HDR | ONE COMPANY *Many Solutions*

– Memorial Bridge

Northeast Cwt Sheave

– Areas of paint failure w/ corr.

Northwest Cwt Sheave

– Banging coming from sheave during lowering operation
– Areas of paint failure

Memorial Bridge Span Features

Performed by: MPK

Date : 5/28

Bumper Blocks

Shaft Mark, Location Description	Exterior Conditon, Bolts and Housing	Remarks
Northeast	All Blocks present Most have light to Med cracks along the grain.	
Northwest		
Southeast		
Southwest		

Counterweight System

Performed by: KC

Date : 5/27 see additional measurements

Counterweight Ropes

				Condition of Wire Rope				
Shaft Mark, Location Description	Exterior Conditon	Rope Diameter Measurements	Lay Length Measure	Broken Wires	Abrasion of Wires	Pitting of Wires	Peening of Wires	Crown Wear
Northeast	Lubrication good							
Southeast	lubrication good							

Memorial Bridge Span Drive

Performed by: MPW

Date : 5/27/08

Span Drive Open Gears

Gear Mark, Location Description	Chordal Addendum	Chordal Thickness		Backlash at full closed		Notes
		Previous	Measured	Previous	Measured	
Gear Northeast	0.645	Original 1.215 Prev. 1.205	T 1.198" M 1.205" H 1.205"	0.110	0.137"	
Pinion East	0.662	Original 1.215 Prev. 1.205	T - 1.093" M - 1.092" H - 1.095"			
Gear Southeast	0.645	Original 1.215 Prev. 1.205	T 1.200" M 1.204" H 1.203"	0.047	0.114"	

Memorial Bridge Span Drive

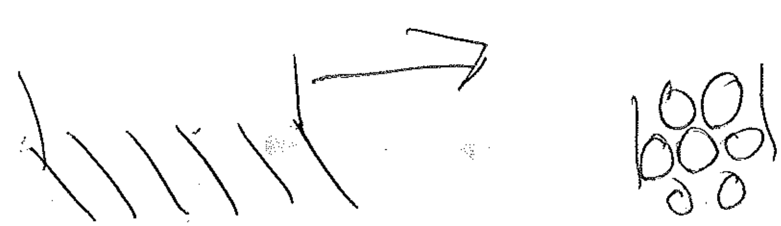
Performed by: MPM, KC

Date : 6/29

Operating Ropes

Shaft Mark, Location Description	Exterior Condition, End Connections	Rope Diameter Measurements	Lay Length Measure	Condition of Wire Rope				
				Broken Wires	Abrasion of Wires	Pitting of Wires	Peening of Wires	Crown Wear
Northwest	Not Accessible for direct inspection							
Southwest	Good lubrication	Raise E 1,018 W 1,025 Lower E 1,012 W 1,025	Raise E 12,830 W 12,660 Lower E 13,087 W 13,085					7/32 Raise

Notes:



Memorial Bridge Span Drive

Performed by: KPM

Date : 5/22/09

Operating Ropes

Shaft Mark, Location Description	Exterior Condition, End Connections	Rope Diameter Measurements	Lay Length Measure	Condition of Wire Rope				
				Broken Wires	Abrasion of Wires	Pitting of Wires	Peening of Wires	Crown Wear
Northeast	Not Accessible for direct inspection							
Southeast	Good lubrication	Raise E 1.031" W 1.014" Lower E 1.028" W 1.024"	Raise E- 12.780" W- 12.700", 12.900" Lower E- 12.800", W- 13.085",					X 7/32" On Raise

Memorial Bridge Span Drive

Performed by: MAK

Date : 5/22/09

Operating Drums

Shaft Mark, Location Description	Exterior Conditon, Bolts and Housing	Rope Groove Measurements	Remarks
Northeast	- Not Accessible for grooves - Good lubrication		
Southeast	- Some light corr. in groove - no defects	1" groove grease tight	

Operating Drums MPK 5/27/09

Shaft Mark, Location Description	Exterior Conditon, Bolts and Housing	Rope Groove Measurements	Remarks
Northwest	Heavily lubricated Not accessible for groove inspection	1" guage tight.	
Southwest	Not Accessible for groove inspection lubrication ok		

Notes:

Memorial Bridge Span Drive

Performed by: MPK

Date : 5/27/09

Drive Machinery

Shaft Mark, Location Description	Exterior Conditon, Bolts and Housing	Remarks
North Motor	- Painted OK - Seals OK - Bolts OK - Grease plugs ok	
South Motor	- All OK, same as North motor	
North Thruster Brake	- All OK, same as south brake	
South Thruster Brake	- links OK, some paint failure - Brake wheel, Pad thickness OK - Thruster & seals OK	

Drive Machinery MPM 5/27/09

Shaft Mark, Location Description	Exterior Conditon, Bolts and Housing	Remarks
North Disc Brake	Painted OK Key + Keyway OK	Nothing more than the exterior condition can bee seen. No manual brake release
South Disc Brake	Hub engaged w/ 4 out of 8 tapped holes and studs.	
Main Reducer	- some light corr. inside. - slight foaming of lube. - sight glass, looks about 1" overfilled - Painting bolts OK. ~~North~~	- Reducer does not have a breather. - North mounting bolts do not fully engage.
Back up Reducer	~~No~~	- Reducer does not have a breather
LPG Motor	- Mounting OK - Transmission may leak.	

- During raise operation, transmission has a loud screeching noise. Possible worn bearings.

Main Reducer hammers through backlash @ bridge stop

Memorial Bridge Span Drive

Performed by: MPM

Date : 5/27/08

Span Drive Open Gears

Gear Mark, Location Description	Chordal Addendum	Chordal Thickness		Backlash at full closed		Notes
		Previous	Measured	Previous	Measured	
Gear Northwest	0.645	Original 1.215 Prev. 1.205	T 1.120" M 1.120" H 1.190"	0.069	0.066"	
Pinion West	0.622	Original 1.215 Prev. 1.205	T-1.220" M-1.190" H-1.200"			
Gear Southwest	0.645	Original 1.215 Prev. 1.205	T 1.184 M 1.180 H 1.184	0.047	0.098"	

Memorial Bridge Span Drive

Performed by: KC

Date : 5/22/09

Leaf Bearings

Bearing Mark, Location Description	Previous Clearance	Measured Clearance	Lubrication	Bolts and Housing	Remarks
B-1 East inboard pinion	orig 0.010 prev. 0.022	N/A	Good	good	Minor surface corrosion Paint failure
B-2 East outbrd pinion	N/A	0.008	Good	Can't see	
B-3 West Inboard pinion	orig 0.010 prev. N/A	.025	Good	Good	Paint failure and [illegible] surface corrosion seal slightly leaks

Memorial Bridge Span Drive

Performed by:

Date :

Leaf Bearings

Bearing Mark, Location Description	Previous Clearance	Measured Clearance	Lubrication	Bolts and Housing	Remarks
B-4 West Outbrd pinion	N/A	0.006	~~good~~ good	~~good~~ can't see	~~Paint failure and corrosion~~
B-5 NE Inbrd drum	orig 0.010 prev. 0.017	.020	good	good	Paint failure and corrosion
B-6 NE outbrd drum	N/A				Behind drum cannot inspect
B-7 SE Inbrd drum	orig 0.010 prev. 0.016	.017	good	good	some paint failure and surface corrosion seal may leak, it was wiped clean
B-8 SE outbrd drum	N/A				Behind drum cannot inspect

Memorial Bridge Span Drive

Performed by:

Date :

Leaf Bearings

Bearing Mark, Location Description	Previous Clearance	Measured Clearance	Lubrication	Bolts and Housing	Remarks
B-9 SW Inbrd drum	orig 0.010 prev. 0.022	.024	good	good	Paint failure minor surface corrosion seal may slightly leak (wiped)
B-10 SW Outbrd drum	N/A				Behind drum cannot inspect
B-11 NW Inbrd drum	orig 0.010 prev. N/A	.025	good	good	Paint failure and surface corrosion seal slightly leaks
B-12 NW Outbrd drum	N/A				Behind drum cannot inspect

Memorial Bridge Span Drive

Performed by: KPK

Date : 5/~~27~~/09
28

*lube lines on all rollers are in poor condition

Operating Rope Sheaves

Shaft Mark, Location Description	Exterior Conditon, Bushing and Housing	Rope Grooves
Northwest Deflector	Areas of paint failure & corr, on sheave pi ed grooves, lube ok could not measure downhaul	wear @ rope groove w/1" guage, uphaul
Northwest Double rollers	Both rollers show signs of wear from rope contact. Areas of paint failure & corr. lube OK	N/A
Northwest Intermediate Sheave	well greased moved freely paint failure ~~on~~ around sheave	grooves have become deeper
Northwest Lower Outer Roller	Rollers are almost completely impacted. corr. in contact area lube OK	N/A
Northwest Lower Inner Roller		N/A

operator reports not all take grease. Supports are questionable

Memorial Bridge Span Drive

Performed by: MPM, KC

Date : 5/28/09

* Operating ropes have scattered areas of Poor lube & corr along the operating length of the rope

Operating Rope Sheaves

Shaft Mark, Location Description	Exterior Conditon, Bushing and Housing	Rope Grooves
Southwest Deflector	Areas of corr on sheave, pin & grooves, lube OK, Fittings OK	uphand rope grooves worn from a 1" gauge
Southwest Double rollers	lower roller does not turn during operation. Signs of heavy wear from rope contact. lube OK	N/A
Southwest Intermediate Sheave	~~Poi~~ paint failure lube OK	outer 2 ~~[illegible]~~ grooves have worn deeper
Southwest Lower Outer Roller	Areas of Paint failure, lube OK slight corr @ contact area	N/A
Southwest Lower Inner Roller		N/A

@ outer roller in all 4 locations, ropes rub against each other @ sheck, and sometimes transpose location on roller.

Memorial Bridge Span Drive

Performed by: MPM, KC

Date : 5/28/09

Operating Rope Sheaves

Shaft Mark, Location Description	Exterior Conditon, Bushing and Housing	Rope Grooves
Northeast Deflector	Areas of paint failure + corr on pins sheaves, and grooves lube OK	worn groove @ uphaul ropes for 1" gage could not measure downhaul
Northeast Double rollers	Roller has wear from contact w/rope, paint failure, lube OK	N/A
Northeast Intermediate Sheave	Paint failure, lube OK	grooves OK
Northeast Lower Outer Roller	Paint failure, corr @ contact area, lube OK.	N/A
Northeast Lower Inner Roller		N/A

Memorial Bridge Span Drive

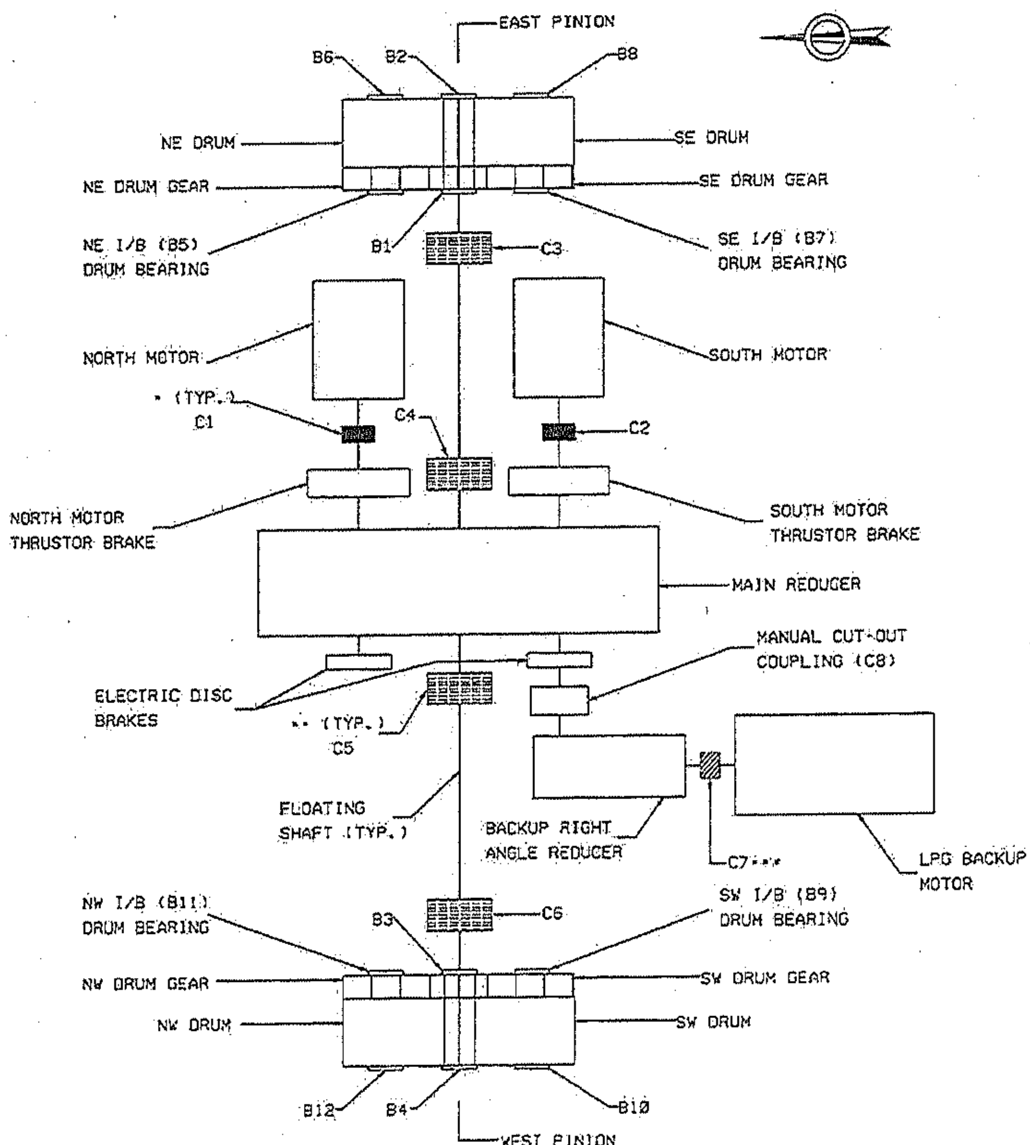
Performed by: MPM, KC

Date : 5/28/09

Operating Rope Sheaves

Shaft Mark, Location Description	Exterior Conditon, Bushing and Housing	Rope Grooves
Southeast Deflector	Areas of corr. on sheave, pins, and groove, lube OK, Fittings OK	uphaul rope grooves slightly worn from 1" gauge
Southeast Double rollers	lower roller does not turn during operation, sign of wear due to rope contact. Areas of Paint failure, lube OK	N/A
Southeast Intermediate Sheave	Lube OK, paint failure	outer 2 groves have worn deeper
Southeast Lower Outer Roller	Areas of Paint failure, lube OK, slight coll @ contact area	N/A
Southeast Lower Inner Roller		N/A

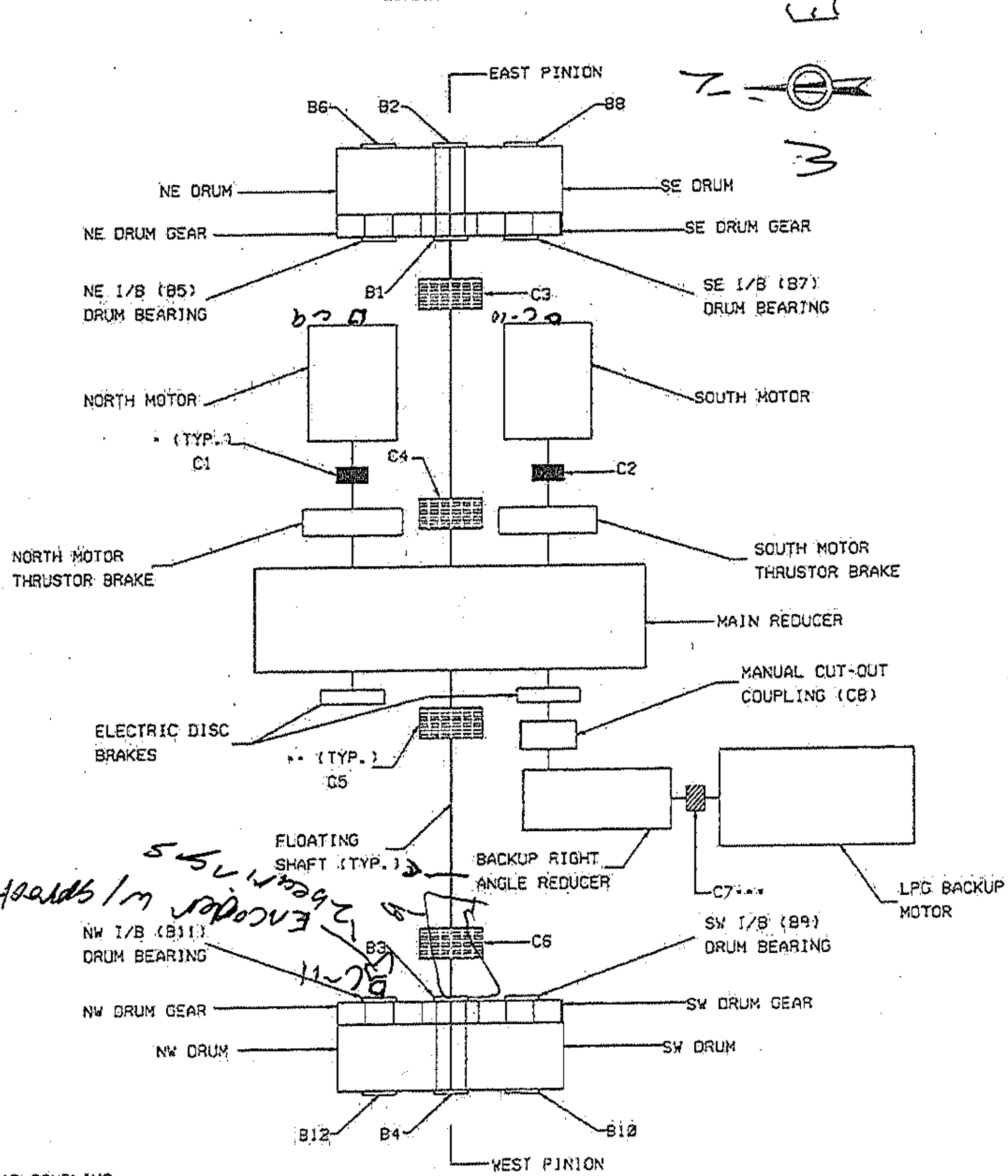
PLAN OF OPERATOR'S HOUSE
(MECHANICAL LAYOUT)



* FULL FLEX GEAR COUPLING
** SEMI-RIGID FLEX GEAR COUPLING
*** GRID COUPLING

C-9, 18 couple to over ~~atat~~ speed, and tach

PLAN OF OPERATOR'S HOUSE
(MECHANICAL LAYOUT)



* FULL FLEX GEAR COUPLING
** SEMI-RIGID FLEX GEAR COUPLING
*** GRID COUPLING

Memorial Bridge Counterweight System

Performed by: KRA

Date : 5/27/09

Counterweight Sheaves

Shaft Mark, Location Description	Exterior Conditon, Bolts and Housing	Rope Groove Measurements	Remarks
Northeast	Areas of pnt failure + corr. - Crack in rim interior on East side of sheave @ 6 o'clock w/bridge fully seated. - light corr in grooves, no other defects.	~~N/A~~ slight wear on 1/2" groove gauge all grooves	
Southeast	- Areas of pnt failure + corr on sheave. - Rope groove condition OK No defects.		- chit equalizer pins creak during operation of span

Counterweight Sheaves KC 6/27

Shaft Mark, Location Description	Exterior Conditon, Bolts and Housing	Rope Groove Measurements	Remarks
Northwest	- Areas of paint failure + corr. - light corr. in grooves, no othe defects	~~N/A~~ Slight wear on 1 1/8" guage for all grooves	- Sheave makes intermittent banging noise during lower.
Southwest	- Areas of paint failure and corr on sheave		- Chatt against pin creek during operation of the span

Notes:

Memorial Bridge Counterweight system

Performed by: KC

Date : 5/27/09

Trunnion Bearings

Bearing Mark, Location Description	Previous Clearance	Measured Clearance	Lubrication	Bolts and Housing	Remarks
Northwest East Bearing 3	N/A	.15, .16, .17	good	some surface corrosion	Paint failure and surface corrosion
Northwest West Bearing 4	N/A	17, 18	11	11	11
Southwest East Bearing		17, 18	11	11	11

note: N Tower Trunnion Bearing caps Do not have ~~a~~ Double cap bolts S Does

Memorial Bridge Counterweight system

Performed by: KC

Date : 5/27/09

Trunnion Bearings

Bearing Mark, Location Description	Previous Clearance	Measured Clearance	Lubrication	Bolts and Housing	Remarks
Southwest West Bearing	N/A	.037	Good	some surface corrosion	paint failure and surface corrosion
Northeast East Bearing 1	N/A	.017, .018	Good	"	"
Northeast West Bearing 2	N/A	.055	Good	Some Surface corrosion	paint failure and surface corrosion
Southeast East Bearing	N/A	.023	"	"	"
Southeast West Bearing		.007	"	"	"

MPM 5/27/09

Leaf Open Gears

Gear Mark, Location Description	General Condition	Lubrication	Keys	Condition of Teeth					
				Normal	Pitting	Rolling - Peening	Abnormal		
							Scoring	Inter-ference	Rust and Cor.
Gear Southeast	Heavy section loss on raise side	OK	N/A		X	X	X		
Gear Northwest	- Greater wear/ damage to raise side		N/A		X	X	X		
Pinion West					X	X	X		

Leaf Open Gears

MPM, KC 5/27

Gear Mark, Location Description	General Condition	Lubrication	Keys	Condition of Teeth					
							Abnormal		
				Normal	Pitting	Rolling - Peening	Scoring	Inter-ference	Rust and Cor.
Gear Southwest	Greater Wear to raise side	OK	N/A		X	X	X		

Memorial Bridge Span Drive

Performed by: MPM, KC

Date : 5/07

Leaf Open Gears

Gear Mark, Location Description	General Condition	Lubrication	Keys	Condition of Teeth					
				Normal	Pitting	Rolling - Peening	Abnormal		
							Scoring	Inter-ference	Rust and Cor.
Gear Northeast	Worst case loading on raise side	OK	N/A		X	X	X		
Pinion East	Heavy section loss on raise side	OK			X	X	X		

Due to heavy wear in gear teeth tooth contact can be felt in bridge shaft being on East side.

Memorial Bridge Span Drive

Performed by: KC

Date : 5/27

I-695 MdTA Inspection Leaf Shafts

Shaft Mark, Location Description	Finish, Keyways, transitions, etc	Remarks
S-1 East Float	minor paint failure	
S-2 West Float	~~[illegible]~~ paint failure where people walk	

Notes:

C-9	Good
C-10	Good condition ~~[illegible]~~ grease on exterior
A	Good cond grease was collected alot of Dust
C-11	Halves are not fully engaged
Encoder	attached to C-11 cover is missing 1/4 screws
Encoder bearing	Paint failure surface corrosion

Memorial Bridge Span Drive

Performed by: KC

Date : 5/27

Drive Couplings

Coupling Mark, Location	Bolts and Housing	Remarks
C-1 North Motor	very little paint chipping Does not follow AAshto	"
C-2 South Motor	minor paint chipping Does not follow AAshto	cannot see seals against brake
C-3 East outbrd Float	good condition Paint failure and minor surface corrosion on bolts	
C-4 East inbrd Float	good condition very little paint failure	leak grease around seal

Notes:

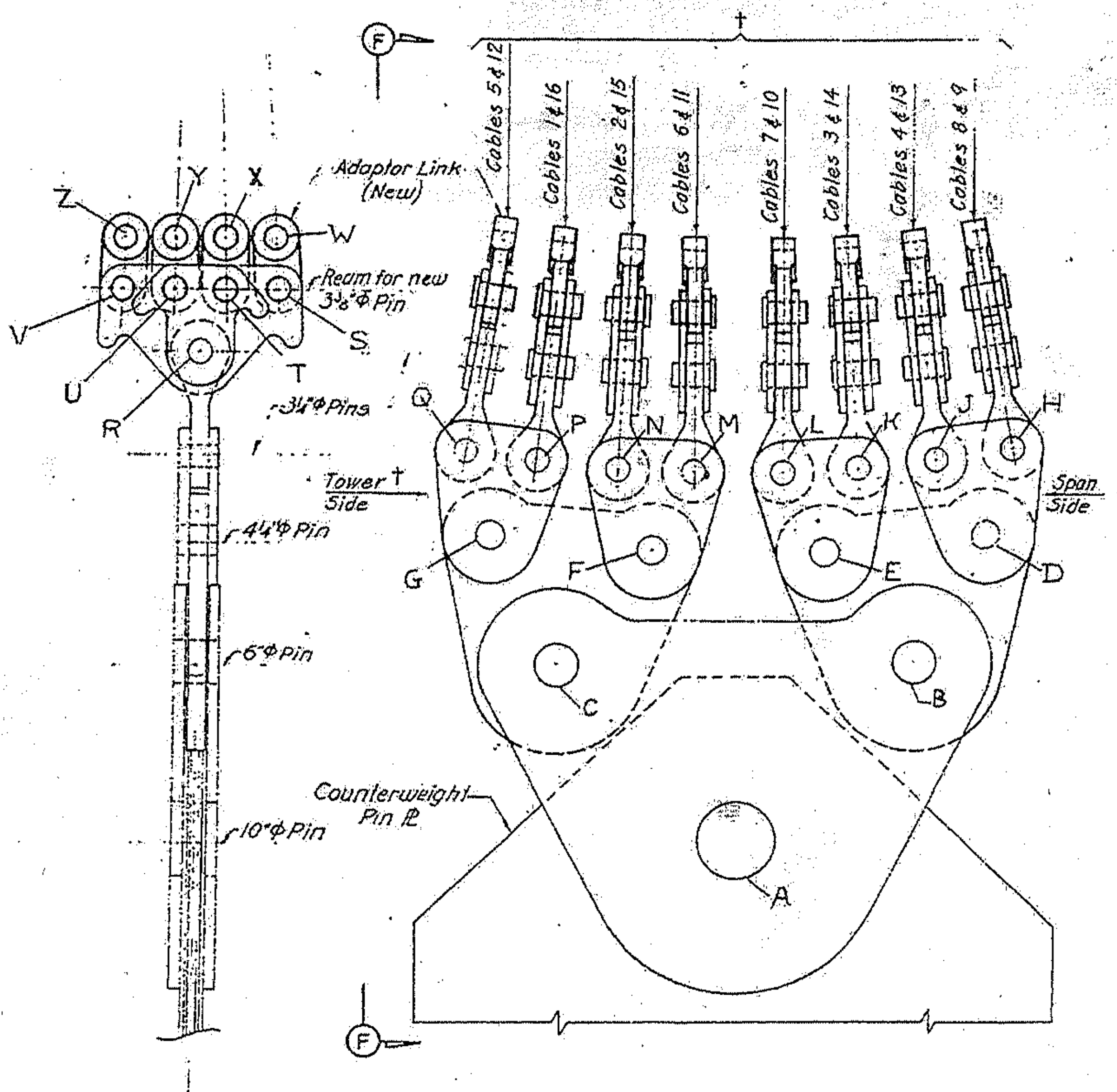
Memorial Bridge Span Drive

Performed by: KC

Date : 5/27

Drive Couplings

Coupling Mark, Location	Bolts and Housing	Remarks
C-5 West inbrd Float	good condition minor paint ~~[illegible]~~ chipping	
C-6 West outbrd Float	Difficult to inspect keys and seals Because of instrument drive, seal leaks a little paint is good	
C-7 LPG Motor	Not painted leaks grease onto guard	
C-8 Clutch Cplg	not all HW painted leaks grease, It appears to be wiped to prevent spraying	



VIEW F-F

EQUALIZER

Memorial Bridge Counterweight System

Performed by: mpm/KC

Date : 6/~~3~~ 4/09

Corner: SE

Counterweight Rope Equalizer Pins

Pin Mark, Location Description	Previous Clearance	Measured Clearance	Pin Connection Condition	Remarks
A Primary				PF, corrosion Fretting corrosion
B Secondary				
C Secondary				
D Tertiary		.008 .018 .019 .020 .021 .022 .023 .024	outside 3/8" Inside	
E Tertiary				

Note pins pop during operation

Memorial Bridge Counterweight System

Performed by: mpm/KC

Date : 6/4/09

Corner: SE

Counterweight Rope Equalizer Pins

Pin Mark, Location Description	Previous Clearance	Measured Clearance	Pin Connection Condition	Remarks
F Tertiary		no measurable Clearance		PF. Corrosion Fretting Corrosion
G Tertiary		19 20 21 22 23 24		
H Fourth level				
J Fourth level				
K Fourth level				

Memorial Bridge Counterweight System

Performed by: KC/mpm

Date : 6/4/09

Corner: SE

Counterweight Rope Equalizer Pins

Pin Mark, Location Description	Previous Clearance	Measured Clearance	Pin Connection Condition	Remarks
L Fourth level				PF Corrosion
M Fourth level				
N Fourth level				
P Fourth level				
Q Fourth level				

Memorial Bridge Counterweight System

Performed by: KC/mpm

Date : 6/~~7~~14/09

Corner: SW

Counterweight Rope Equalizer Pins

Pin Mark, Location Description	Previous Clearance	Measured Clearance	Pin Connection Condition	Remarks
A Primary				PF Corrosion Fretting corrosion
B Secondary				
C Secondary				
D Tertiary		3/8"		
E Tertiary				

Memorial Bridge Counterweight System

Performed by: KC/mpm

Date : 6/4/09

Corner: SW

Counterweight Rope Equalizer Pins

Pin Mark, Location Description	Previous Clearance	Measured Clearance	Pin Connection Condition	Remarks
F Tertiary		no clearance observable	~~[illegible]~~ Pops as it rotates	PF corrosion Fretting corrosion
G Tertiary				
H Fourth level				
J Fourth level				
K Fourth level				

Memorial Bridge Counterweight System

Performed by: KC/MPM

Date : 6/4/09

Corner: SW

Counterweight Rope Equalizer Pins

Pin Mark, Location Description	Previous Clearance	Measured Clearance	Pin Connection Condition	Remarks
L Fourth level				PF corrosion
M Fourth level				
N Fourth level				
P Fourth level				
Q Fourth level				

Memorial Bridge Counterweight System

Performed by: u

Date : 6/17/09

Corner: NE

Counterweight Rope Equalizer Pins

Pin Mark, Location Description	Previous Clearance	Measured Clearance	Pin Connection Condition	Remarks
A Primary				PF corrosion Fretting corrosion
B Secondary				
C Secondary				
D Tertiary		3/4"		
E Tertiary		~~.0615~~ .002 ~~[illegible]~~	.0025 .003 .01+.025	

Memorial Bridge Counterweight System

Performed by: KC / mpm

Date : 6/4/09

Corner: NE

Counterweight Rope Equalizer Pins

Pin Mark, Location Description	Previous Clearance	Measured Clearance	Pin Connection Condition	Remarks
F Tertiary				PF Corrosion Fretting Corrosion
G Tertiary		9/18"		
H Fourth level				
J Fourth level				
K Fourth level				

Memorial Bridge Counterweight System

Performed by: KC/mpm

Date : 6/4/09

Corner: NE

Counterweight Rope Equalizer Pins

Pin Mark, Location Description	Previous Clearance	Measured Clearance	Pin Connection Condition	Remarks
L Fourth level				PF/~~corro~~ corrosion
M Fourth level				
N Fourth level				
P Fourth level				
Q Fourth level				

Memorial Bridge Counterweight System

Performed by:

Date : 6/4/07

Corner: NW

Counterweight Rope Equalizer Pins

Pin Mark, Location Description	Previous Clearance	Measured Clearance	Pin Connection Condition	Remarks
A Primary				PF Corrosion fretting corrosion
B Secondary				
C Secondary				
D Tertiary		3/16"		
E Tertiary		cannot measure clearance	[illegible]	

Memorial Bridge Counterweight System

Performed by: MPM

Date : 6/4/09

Corner: NW

Counterweight Rope Equalizer Pins

Pin Mark, Location Description	Previous Clearance	Measured Clearance	Pin Connection Condition	Remarks
F Tertiary		22 33 24 18		PF corrosion fretting corrosion
G Tertiary	≈ 1/2" 9/16"	more than the entire feeler gage		
H Fourth level				
J Fourth level				
K Fourth level				

Memorial Bridge Counterweight System

Performed by: KC/MPM

Date : 6/4/09

Corner: NW

Counterweight Rope Equalizer Pins

Pin Mark, Location Description	Previous Clearance	Measured Clearance	Pin Connection Condition	Remarks
L Fourth level				PF/Corrosion
M Fourth level				
N Fourth level				
P Fourth level				
Q Fourth level				

1-4 SW Corner
5-8 NW Corner
9-12 SE Corner
13-15 NE Corner
16-17 NW corner of memorial

Sarah Long

Sarah Long

Southwest Corner

SW

N ↓ 20 0 0 0 0 0 0

8 14 16

1 0 0 0 0 0 0 0

7 13 15

15 Cycles

Rope Trial	#1	2	3
1	5.87	~~6.10~~ 5.97	5.81
2	5.85 ~~[illegible]~~ ~~[illegible]~~	5.57	5.69
3	5.63	5.75 ~~6.02~~	5.75
4	~~6.10~~ 5.32	~~5.94~~ 5.43	5.82
5	5.69	6.16	~~5.37~~ 5.72
6	5.04	5.09	4.87
7	5.92	5.72	5.68
8	5.34	6.40	5.59
9	6.22	5.72	~~5.46~~ 6.13
10	6.31	5.57	5.78
11	6.13	5.56	5.78
12	5.50	5.56	5.73
13	5.19	5.41	5.28
14	5.16	5.56	5.16
15	5.91	5.78	5.94
16	9.99	10.19	10:00

Portsmouth, NH

10/14: 8am 31 deg. F, 3pm 50 deg. F

10/15 8am 28 deg. F, 1pm 42 deg. F

#16 7.44F, 7.44F, 7.37 F

#14 8.40F, 8.37F, 8.38F

Not Touching guide

Sarah Long

Northwest Corner

N ↑

16 12 8 4
0 0 0 0 0 0 0 6 2
0 0 0 0 0 0 0 6 1
5 1 3

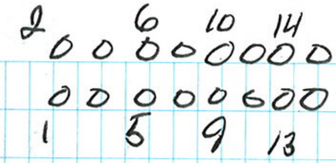
Rope	#1	2	3
#16	4.63	4.60	4.81
#14	4.75	4.78	4.78 #1
2 -	10.00F	9.97F	10.00F
3	9.31F	9.31F	9.34F
4 -	8.00F	7.81F	7.85F
*5	10.72F	10.59F	10.63F
6 -	8.21F	8.19F	8.25F
7	10.47F	10.28F	10.32F
8 -	8.43F	8.40F	8.53F
9 -	10.25F	10.28F	10.28F
10 -	5.06 9.00 (F)	8.85 (F)	8.78
11	9.50F	9.69F	9.56F
12	4.46 (1/2)	3.69 (F)	8.97 (F)
13	9.22F	9.47F	9.34F
#1	8.84F	8.86F	8.75F
#15	9.75F,	9.53,	9.62F

* Not Against Guide

Southeast Corner

Sarah Long

↑ N



1 -	9.44	9.47	9.46	
2 -	8.37	8.47	8.47	
3	7.78	7.66	7.94	
4	8.22	8.15	8.28	
5	8.06	8.06	8.09	
6	8.93	8.82	9.01	*
7	7.57	7.57	7.59	↑ Not
8	10.00	9.94	10.00	Touching
9	10.16	10.18	10.28	Guide ↓
10	9.97	10.09	9.94	*
11	8.87	8.88	9.02	
12	9.19	9.10	Wind	
13	8.47	8.60	8.57	
14	8.44	8.37	8.50	

* Not Touching Guide

*15	7.75	7.69	7.68
16	10.69	10.53	10.53

Northeast Corner

Sarah Long

N

2 6 10 14

o o o o o o o o

o o o o o o o o

1 5 9 13

1 -	10.53,	10.63	10.53	
2 -	8.28,	8.28,	8.29	
3 -	9.41,	9.44,	9.50	
4 -	7.80,	7.47,	7.40	
5 -	9.32,	9.31,	9.40	
6 -	8.15,	8.18,	8.25	*
7 -	10.34,	10.44,	10.41	
8 -	8.12,	8.16,	8.25	*
9 -	10.06,	10.03,	10.00	
10 -	8.47,	8.47,	8.50	*
11 -	9.60,	9.47,	9.31	
12 -	8.32,	8.34,	8.40	
13 -	9.72,	9.53,	9.56	
14 -	7.31,	7.38,	7.34	
15 -	8.94,	8.94,	8.91	
16 -	8.82,	8.91,	8.94	

Not in contact w/ frame.

ELECTRICAL STUDY

1 EXECUTIVE SUMMARY

The Memorial Bridge Movable Span Electrical facilities were inspected on May 27th and 28th of 2009. The electrical inspection team examined all the accessible components of the bridge electrical and control systems. The bridge electrical system was found to be nearing the end of its reliable service life and many of the system components have become obsolete with unavailable spare parts. Three levels of rehabilitation scope were considered, described as follows: (1) Minimum Rehabilitation which represents the minimum repair recommendations required to resolve the findings that affects the bridge reliability and general public and maintenance staff safety and should be resolved in a timely manner, (2) Moderate Rehabilitation which represents repair recommendations that will further increase the electrical and control system reliability, (3) Major Rehabilitation which represents repair recommendations that will make the bridge electrical and control systems very safe and reliable as per the current design standards. Recommendations for each level of rehabilitation are presented in Section 5.

2 DESCRIPTION OF SYSTEMS

This bridge is a span drive vertical lift bridge that carries a single roadway deck of US Route 1 between the towns of Portsmouth, NH and Kittery, ME.

The bridge is manned 24/7 year round and is operated from the machinery control house located on the top of the movable span. A gate tender is stationed on each end of the bridge to manually operate a pair of structural steel swing gates. In addition to the manually operated gates, there are two remotely operated warning gates located at the abutments. There are curtain style barrier gates located at the approach spans. The barrier gates are abandoned in place. The remote operated gates are controlled from the gate tenders house near each abutment.

The utility power feeder is supplied from the south end of the bridge from Portsmouth, NH. The power is transferred to the movable span thru vertical catenaries on the south tower.

Bridge back up power is supplied through a diesel two cylinder Onan, air cooled propane generator to run the hotel loads and a propane operated motor is used to lift the movable span in case of a power failure. A pair of magnetic DC brakes powered off the propane motor DC system is used for braking during the emergency bridge operation. There is no backup power for the gates. The heat rejected from the engines is rejected into the control house. Doors are opened on each end to allow the heated air to escape. The propane tanks are located at the roadway level in a fenced enclosure.

Two 100HP motors with a single SCR thyristor drive are used to operate the span. Both motors are controlled by the same drive and they are both required to run in order to lift the span. An over speed switch and a tachometer generator are installed on the back end of each motor to provide the feedback required by the drive. The tachometer is switch selected providing redundancy.

There are no span locks on the bridge. The operator pulls tension on the downhaul ropes and then applies the brakes to hold the span closed.

Other ancillary systems such as navigation, access and egress lighting are located at various locations about the bridge.

The bridge electrical and control system was found to be operational but the reliably of the system is questionable. Some of the system components have become obsolete. Finding a retrofit replacement in an emergency situation may be challenging and require an extended period of operating with the backup engine. The components most susceptible to failure with prolonged outage are the brake thrusters, thyristor drives and the rotary cam limit switch.

3 INSPECTION APPROACH AND METHODOLOGY

A visual inspection of the bridge electrical and control systems components was conducted. All the accessible cabinets and enclosures were opened and inspected. Tests consisting of insulation resistance and load current measurements were also conducted. Tests were performed to highlight areas that could be problematic towards the continued safe and reliable operation of the bridge as well as its effectiveness in supporting both roadway and marine traffic operations.

4 INSPECTION FINDINGS

4.1 Main Electric Service

There is one utility feeder supplying power to the bridge from Portsmouth NH at the south end of the bridge. The system voltage is 480/277V three phase. A separate feed supplies the equipment at the north tower and equipment on the approach span from Kittery, Maine. The main service disconnect switch for the bridge is located on the south abutment and feeds power to the exposed bare bronze catenaries on the face of the tower. The power is then transferred to the movable span through spring loaded bronze rollers that are in contact with the catenaries, these rollers remain in contact with the catenaries as the span is moving. The main disconnect switch does not have the clearance required to safely access or operate the switch. Any rehabilitation should include the relocation of the disconnect to a new location that meets NEC code for clearances around electrical equipment.

The catenaries are problematic since they are an electrical hazard to trespassers, or authorized persons performing maintenance or inspection on the bridge. The catenaries also ice up in the winter. Local switching is required to provide heating power from the deicing transformers that are installed on the tower at the south end of the bridge at the top chord of the truss to melt the ice (See photo E-1). This process must be performed in the icy environment that has an elevated risk of accidents for the maintenance and operation crew. As a part of a moderate or major rehab the catenary system should be replaced with a droop cable system. The droop cable system will require less maintenance, enhance the reliability of the electrical system and minimize the risk of accidents. Installing droop cables on both towers would provide the means for the gates and traffic signals to be powered from the emergency generator. The droop cables shed any ice accumulation as they flex when the span is moved.

4.2 Back up Electric Service

The back up power is provided from an air cooled 4KW two cylinder Onan propane fired engine generator set. This generator is sized to supply back up power to some of the hotel loads in the machinery and control room that are powered normally from a branch circuit in the 120/208V lighting panel. The loads include the span height indicator. None of the normal bridge control devices are powered from this engine. An Automatic Transfer Switch (ATS) is used to switch the branch circuit when the main power feed is lost or catenary deicing is required. All the bridge hotel loads on the single branch circuit are connected directly to the load side of the ATS. The ATS is an obsolete model that uses magnetically held contactors. During the HDR inspection, the ATS failed to switch when required. The electrical service contractor was called out to perform repairs. Any rehabilitation approach should include the replacement of the existing ATS with a modern electrically switched mechanically held transfer switch in order to increase the system reliability.

A propane powered, four cylinder, spark ignition engine is used to operate the span when the main power feed is lost. A gear type manual clutch is used to connect the output shaft of the engine reducer to the primary gearbox. Limit switches are provided to disable the main control system to assure that the engine reducer is not back driven by the main span motors. During emergency operation the thruster brakes are hand released and the DC powered magnetic brakes are used for braking. In case of an emergency where the DC powered magnetic brakes fail to set; the operator will have to leave the control station to manually set the thruster brakes. Any design rehabilitation approach should include the upgrade of the generator to a three phase unit with sufficient capacity to allow the thruster brakes to be electrically operated when the backup engine is being used. This upgrade will enhance the reliability of the electrical system and enhance the safety of the operators and personnel in case of the failure of the DC magnetic brakes. As part of a major rehabilitation, a substantially larger generator with the appropriate power system equipment should be installed to eliminate the propane engine and provide for normal operation of the span drive. The generator and ATS could be located on shore if a droop cable system is installed. There is sufficient real estate for the equipment. As part of any rehabilitation approach, the usage of the propane fueled engine in the operator house requires carbon monoxide detectors to be installed to alert the operator of any exhaust or plenum leaks (See photo E-2). A CO detector is generally required where gas fired space heaters are installed as is the case in the Memorial Bridge control house.

4.3 Motor Control Centers (MCC)

The MCC is located in machinery and control room and is fed directly from the catenary system located at the south tower. The MCC is 1950's design and requires personnel to open the doors to manipulate breakers with exposed bare copper bus and other exposed, energized components. (See photo E-3). This practice is unacceptable by today's standards since it exposes the electrician to the arcing faults that might result during a switching process. Any rehabilitation approach must include provisions to de energize the MCC without opening the door or replace this MCC with a modern dead front style MCC where the switching is performed without personnel being exposed to energized conductors.

4.4 Motors and Drives

The span is raised and lowered using two 100HP, 585RPM, 460V, 3 phase, 60 cycle, wound rotor type motors. These motors are controlled by a single Westinghouse Silicon Controlled Rectifier (SCR) thyristor drive that is located in the machinery room MCC. Both motors are connected directly to the load side of the drive through independent three phase breakers. Resistor grids are connected to motor secondary windings and are used to modify the speed torque characteristics of the motors The inspection found the motors brushes to be in serviceable condition, however the motors have excessive grease inside which should be cleaned as part of regular maintenance. It was also identified that the lead wire insulation on the secondary circuit of the north motor was chaffing on the motor frame. The electrical service contractor was requested to provide corrective action. Temporary measures were taken to minimize the potential for the insulation on the rotor leads to chafe through and short out as they leave the motor rotor housing by adding rubber insulation (See photo E-4). More permanent measures should be taken.

The motor secondary resistor grids are mounted in the machinery room next to the MCC. The secondary resistors have limited guards in place. Non grounded parts and conductors are exposed (See photo E-5). Guard should be installed to completely cover the resistors.

The motor primary and secondary windings were megger tested during the inspection and the insulation resistance values were found to be 550 Megohm for all the windings. These values are acceptable and the motors windings appeared to be in good condition.

With the current drive configuration, one thyristor drive is used to run both motors. If this thyristor drive fails, there will be no means to operate the bridge electrically. Circuit board level parts for this drive are no longer available from the Original Equipment Manufacturer (OEM). With the advent of modern digital motor drives, there are few technicians familiar with this technology should repairs be required. A minimum rehabilitation scope should include the installation of a new redundant thyristor drives to enhance the reliability of the system. The motors should be cleaned and serviced by a qualified motor shop to replace the secondary circuit extension leads. The motor secondary resistor grids should be replaced with modules that have factory designed and installed guards to minimize the hazard of the exposed energized conductors. A moderate and major rehabilitation scope should include the replacement of the span motors and installation of modern flux vector drives.

4.5 Brakes

The bridge braking system consists of two thruster actuated machinery brakes and two Direct Current (DC) electro-magnetic disc brakes. The thruster brakes are mounted on the output shaft of each motor and the electro-magnetic disc brakes are mounted on the opposite end of the input shaft to the reducer. The thruster brakes are used when the bridge is operated using the electrical power and the DC electromagnetic brakes are used when the bridge is operated on the emergency engine.

The electromagnetic brakes are powered from the DC system of the emergency motor and they appeared to be in a good condition. The emergency motor self charges the batteries when it is running.

The thrusters are 460V, 3 phase units and even though they appeared to be in a good condition they are an obsolete GE model. Spare parts, such as seals, are not available (See photo E-6). Both brakes must be released, electrically or by hand to operate the span using the electric motors or emergency engine. A design provision is that only one thruster brake can be hand released and still operate the electric drive. The brake thrusters were megger tested and the insulation resistance was found to be 550 Megohm or greater for each. These values are acceptable and the brakes appeared to be in a good condition. Two lever arm limit switches are mounted on each thruster brake for brake released and brake hand released indication. These switches appeared to be in a good condition. A moderate and major rehabilitation scope should include the replacement of the existing thruster brakes with modern single piston thruster brakes and the installation of new limit switches.

4.6 Control Desk

The main control desk is located in the machinery and control house. The desk has seen several modifications over the years and a new console was added next to the original control desk for gates and lights. The modifications have made the controls separated and not laid out in a logical order that follows the sequence of the bridge operational procedures. The gates and traffic lights on each abutment are actually controlled from a control panel that is located inside the gate attendant shacks and the original controls on the add-on console are bypassed. Any rehabilitation approach shall include the replacement of the control desk with a human factors designed integrated control desk that provides for all bridge operating controls at the main control desk with provisions to operate the gates from remote control stations should the need arise. An industrial quality radio communication system with redundant links should be used to communicate with local gate controls. In addition, a significant amount of video equipment is clustered near and on the control desk. This equipment should be segregated in to a separate cabinet.

4.7 Gates And Traffic Signals

The bridge has electrically operated warning gates at the abutments, and electrically operated curtain gates at the towers. The bridge staff indicated that the curtain gates do not work in cold weather and have been abandoned. One gate attendant on each end of the movable span manually operates a pair of structural steel swing gates. The gate attendants have to be onsite for any bridge opening. The structural steel swing gates may not conform to current requirements for barrier gates and any rehabilitation approach should include the replacement of these gates with AASHTO compliant barrier gates rated for this application.

Attendant shacks are provided for the gate attendants and a traffic signals and gates control console is located inside these shacks. The traffic signals and gates controls are not interlocked with the main bridge controls. The lack of interlocks requires a high level of vigilance on the part of the gate tenders and bridge operators. A human error might lead to incidents such as the movable span being lifted before the gates are closed and locked. Providing a data link for remote indication and control for the gates and traffic signals to the control system will allow the operator to operate the bridge without relying on any other staff. This option will decrease the labor cost associated with the bridge operation and will enhance safety by providing built in interlocks and eliminating the presence of the gate attendant in the roadway. This recommendation should be included with any rehabilitation approach that involves the bridge control system.

4.8 Limit Switches

Field limit switches are provided for span closed, normal open and full open. The field wiring for the new span seated limit switches uses THWN wire. THWN wires are not compliant with AASHTO requirements. Over the long term the insulation wears through due to vibration and causes short circuits. A major or moderate rehabilitation should include the installation of RHW type wire for the limit switches wiring in conformance with the latest guidance provided in the 2008 interim revisions to the AASHTO LRFD Movable Highway Design Specifications section “8.9 Electrical Conductors”.

4.8.1 Span Seated:

The movable span is provided with one heavy duty, plunger operated, limit switch on each end of the span that is tripped by a structural steel target mounted to each rest pier to provide span seated indication (See photo E-7). The switches have been recently replaced and are in a very good condition. From field experience the plunger switches can be problematic. If they are not adjusted properly, they may be jammed and will no longer operate reliably. It is recommended as part of a moderate or major rehabilitation to replace the existing plunger switches with proximity sensors. Proximity sensors are a more reliable since they do not need to make physical contact to function.

4.8.2 Span Position:

The bridge has one multi turn, chain driven lead screw rotary cam limit switch. This switch is the main input used for the bridge control system. It is an obsolete model and will be difficult to find repair parts or to replace the switch in case of a failure. From a functional stand point this switch is difficult to adjust (See photo E-8). It is recommended as part of a moderate or major rehabilitation to replace the lead screw rotary cam limit switch with a modern state of the art rotary cam limit switch.

4.8.3 Full Open and Over Travel:

There are several lever arm limit switches on each corner near the top chord of the truss. One switch at each corner is used to provide the full open indication and one switch is used for over travel indication. The existing switches did not appear to be functional and some of them were missing levers (See photo E-9). Functional over travel and full open limit switches are essential components in the bridge control system that enhance the safety and reliability of the bridge control system. The installation of new full open and over travel lever arm limit switches should be included in the scope of any rehabilitation approach. A control system upgrade should trigger a complete review of limit switch requirements, including limit switches for use during emergency engine operation.

4.9 Navigation Lights

Navigation lights are installed on the piers and on the north and south face of the lower truss over the navigation channel of the movable span to provide visual guidance for the marine traffic. Several junction boxes are tied to the bridge with wire and several access doors on the navigation lights are open due to missing or defective closing hardware (See photo E-10). Operational and reliable navigation lights are important indications to mariners and are required to be installed and be maintained by US Coast Guard requirements. It is recommended that as a part of any rehabilitation scope the lighting fixtures, supports and raceways be rehabilitated A major rehabilitation scope should include the installation of new navigation lighting system.

Aircraft warning beacons are installed on the top of the towers. As part of any rehabilitation, the beacons should be relamped with energy efficient, long life LED lamps. A major rehabilitation should trigger the replacement or the entire aircraft warning beacon with FAA compliant units.

4.10 Miscellaneous

The 480V distribution panel board and the lighting transformer inside the machinery and control room appears to be original bridge equipment, and items such as closure plates on the face of the breaker panels are missing. The equipment appears to be beyond it normal anticipated 40 year life and should be replaced to assure continued reliable service. Circuits in the new panels should be designed to provide proper coordination, selectivity and segregated to assure that tripping of breakers for non critical loads do not interrupt critical functions.

5 RECOMMENDATIONS

5.1 Minimum Rehabilitation:

1. Main electric service
    a. Install new main disconnect switch in an accessible location with the appropriate access and work clearances on the south pier.
2. Back up electric service
    a. Replace the ATS
    b. Install carbon monoxide detection system in the machinery room
    c. Upgrade the generator to provide additional power required to operate the thruster brakes when bridge is run on the emergency engine.
3. Motor control centers
    a. Install a new main deadfront disconnect switch in the operators house.
4. Motors and drives
    a. Replace existing motor secondary resistor grids with modules that have a factory designed and installed guards.
    b. Install a two thyristor drives for the span motors to ensure reliability.
    c. Replace the drive motor rotor extension leads and add bushings or other protection where they pass through the motor frame. Replace motor brushes and clean motor interiors.
5. Brakes
    a. Provide remote control for thruster brakes next to the magnetic brake controls for use during operation with the emergency engine.
6. Control desk
    a. Replace the control desk with a human factors designed integrated control desk, incorporating the gate control stations
7. Traffic gates
    a. Install AASHTO compliant barrier gates
    b. Provide data link and remote communication and control modules coordinated with the bridge control system to allow gate to be operated from control house and interlocked with the control system.
8. Limit Switches
    a. Replace the over travel and full open lever arm limit switches
9. Navigation lights
    a. Rehab or replace navigation lighting that is missing functional access door closure hardware or has degraded raceway system
10. Miscellaneous
    a. Provide instrument rack and relocate all video equipment from the control desk

5.2 Moderate Rehabilitation:

1. Main electric service
    a. Install new main disconnect switch in an accessible location with the appropriate access and work clearances on the south pier.
    b. Replace catenary system with a droop cable system.
2. Back up electric service
    a. Replace the ATS
    b. Install carbon monoxide detection system in the machinery room
    c. Upgrade the generator to provide additional power required to operate the thruster brakes when bridge is run on the emergency engine.
3. Motor control centers
    a. Replace MCC with a modern dead front MCC.
4. Motors and drives
    a. Replace span motors and install new flux vector drives.
5. Brakes
    a. Provide remote control for thruster brakes next to the magnetic brakes controls by the back up emergency engine
    b. Replace thruster brakes with modern brakes
    c. Install new brake limit switches
6. Control desk
    a. Replace control desk with human factors designed integrated control desk and integrate the remote gate control stations
7. Traffic gates
    a. Install AASHTO compliant barrier gates
    b. Provide data link and remote communication and control modules coordinated with the bridge control system to allow gates to be operated from the control house and interlocked with the control system
8. Limit Switches
    a. Replace the over travel and full open lever arm limit switches
    b. Replace existing span seated plunger switches with proximity sensors
    c. Replace lead screw rotary cam limit switch with modern state of the art rotary cam limit switch
    d. Replace the THWN wires used for devices wiring with RHW wire.
9. Navigation lights
    b. Rehab or replace navigation lighting that is missing functional access door closure hardware or has degraded raceway system
10. Miscellaneous
    a. Provide instrument rack and relocate all video equipment from the control desk
    b. Replace lighting transformer and lighting panel board. Rewire operator's house to provide selectivity between circuits.

5.3 Major Rehabilitation:

1. Main electric service
    a. Install new main electrical service with droop cable system
2. Back up electric service
    a. Install new emergency generator and ATS capable of electrically operating the bridge in case of a power failure.
3. Motor control centers
    a. Install new motor control center
4. Motors and drives
    a. Install new flux vector drives and motors
5. Brakes
    a. Install new thruster brakes with modern single piston thruster and new limit switches
6. Control desk
    b. Replace control desk with human factors designed integrated control desk.
7. Traffic gates
    a. Install AASHTO compliant barrier gates
    b. Provide data link and remote communication and control modules coordinated with the bridge control system to allow gates to be operated from the control desk in the control house or locally
8. Limit Switches
    a. Install new limit switches
9. Navigation lights
    a. Install new navigation lighting system
    b. Replace the aircraft beacon system with FAA compliant system

Appendix A
Photos

Photo E-1: Catenaries are problematic since they ice up in the winter and local switching is required to provide heating power from the transformers that are installed on the top of the bridge truss level at the south end to melt the ice

Photo E-2: The use of propane fired emergency engine in the machinery and control room requires the installation of carbon monoxide detectors to alert the operator of any exhaust or plenum leaks

Photo E-3: The MCC is of obsolete design with exposed bare copper bus which exposes the switch person to the arcing that might occur during the switching process

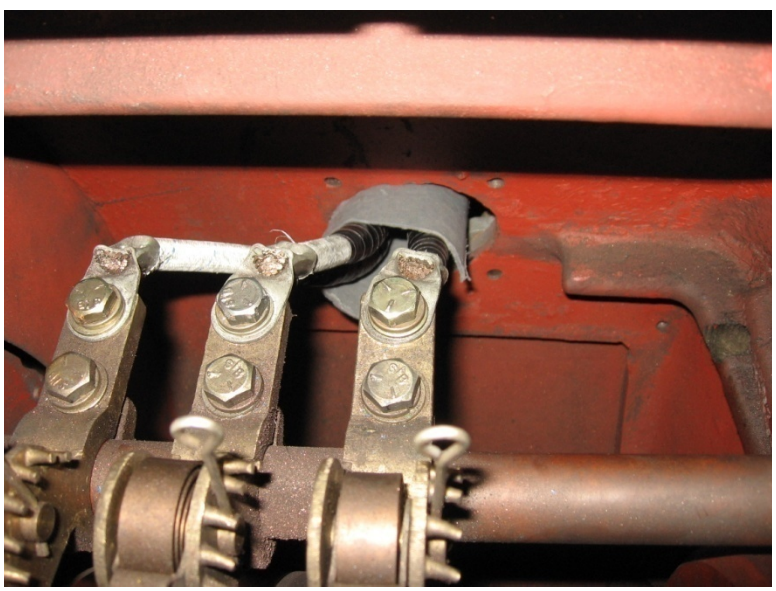


Photo E-4: Temporary measures were taken to minimize the potential for the insulation on the rotor leads to wear through and short out as they leave the motor rotor housing

Photo E-5: The secondary resistors have exposed energized elements and conductors and the existing guard does not fully encompass for the hazard

Photo E-6: Even though the thruster brakes appeared to be in a good condition, they are an obsolete GE model and no spare parts, such as seals, are available

Photo E-7: From field experience the plunger switches for the span seated indication are problematic. If they are not adjusted properly, they get damaged easily.

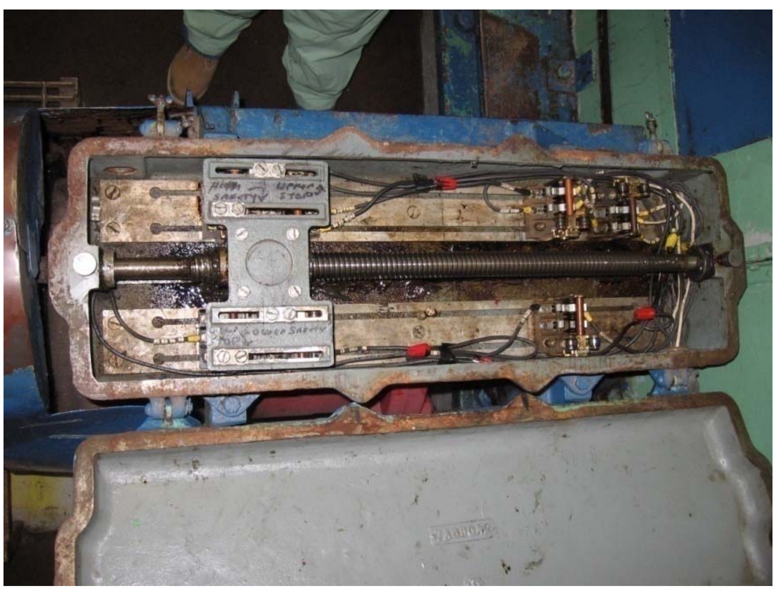


Photo E-8: The lead screw rotary cam limit switch is an obsolete model and it is very difficult to adjust

Photo E-9: Many of the existing lever arms limit switches normally used for span full open and over travel indication switches are not functional

Photo E-10: Several access doors on the navigation lights are open due to missing or defective closing hardware

Appendix B
Field Inspection Sheets

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Project: NH Bridge	Computed:	Date:	
Subject: Memorial Bridge	Checked:	Date:	
Task:	Page:	of:	
Job #:	No:		

1) Utility Supply: — Cable & Conduit faulted

2 Service Equipment — located at ~~the~~ pier could not be inspected because of access. Meter access is on Utility Pole

3. Genset / ATS — ATS has fault with the mechanical contactors and doesn't pick up.
Genset makes a huge noise halfway through the lifting exercise

4. MCC — reliability
Integrated systems with professional control desk ~~that~~ a new operator can be trained properly

5. Panelboards & Transformers — heating Boards missing front covers.
replacement parts may be hard to obtain

6. Festoon / Droop Cables — no festoon / dropp cables, abandoned festoon rail & disconnected communication utilities.
Bridge has Catenaries — Problem — no insulation
Power comes up on the Catenaries, it could ice up

7. Control System — Position Indicators — south gate operators
— 1 span seater, at each end

Project: NH Bridge		Computed:	Date:
Subject:		Checked:	Date:
Task:		Page:	of:
Job #:		No:	

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Field Equipment

— limit Switches – span seaters ok
fully Closed/span seated
~~Rotary~~ Cam/ACME
fully Open
Overtravel – 4 switches, 2 are working, 2 aren't working

— Brakes: thrusters & limit switches are Okay
works fine.

— ~~Gates~~ Span locks – None
motor & limit switches

— Drive motors – notification of Potential Problem
repaired on the spot.
No visual problems with brushes

Projects (continued)

NH DOT Bridge Inspection
27 May 09
Cool, Rainy.

7:00 AM MTG @ NHDOT HQ
Memorial Bridge
Phil - Bridge Operator
Bob - Supervisor
8:50 First Lift
8:55 Matt, Kevin, Jane - North Tower
65' Horizontal Point

Concerns About wind load + Ice load.
Currently Gates - problem with Limit switches - Run In Bypass

NHDOT Mem Br. (2)

South Motor (W)

FR 581-Z 3φ60cy

100 Hp 585 Rpm

460V 100% Load

163A 1 Hr 80°C

Style 76F52586

SH 15-77

Rotor 354V 141 A

Variable speed Type CIP

North Motor

Style 76F52585

Tach Gen

MN 5PY59JY2

Max Spd 5000 Rpm

50.8V DC @ 1000 Rpm

8 x 19" Brake Wheels

9:45 Left 2

Team - move North to South Tower

10:00 Noticed Pair of Secondary Cable Rubbing Problem on North motor

In Control House

71°F 41% Humidity

10:30 > Left 3

10:35 Team off Towers

GE BRAKE
MN IC 9516466T
CAT# 0116B143
POUND FT 1600 MAX
60 MIN
SPRING LENGTH 6-13/16 IN
1600 # THRUSTOR
VOLTAGE 230/460 60 CY
3 ϕ
1-7/8 STROKE
P 55197

THRUSTOR - SPARE
IC 9504 $\boxed{475?}$
215L71909
1600 #
3" STROKE
1/2 HP

71°F
40% RH

MEGGER TEST. PRIMARY
NORTH DRIVE MOTOR
550 MΩ @ 525V
SOUTH DRIVE MOTOR
550 MΩ @ 525V

MEGGER TEST SECONDARY
NORTH DRIVE MOTOR
550 MΩ @ 525V
SOUTH DRIVE MOTOR
550 MΩ @ 525V

BRAKES
NORTH BRAKE THRUSTOR
550 MΩ @ 525V
SOUTH BRAKE THRUSTOR
550 MΩ @ 525V

JOG THE OPERATOR ON BOARD
@ 12:00

TS

OK   OK   OK

PIER US

TS. OK   OK

OK OK.

SPAN SWITCH LS
1 - N
1 - S > OK. THEN WORK.

4PM LEFT.

BRAKES .73-.83 Amps

ALL MOTOR PHASES IN SAME
REGION 185 - 70A

SPAN CHANGER SIGNAL -
RED TO GREEN E&W

THURSDAY 28 MAY 09
RAINING
1ST LEFT @ 9:45

2/2 HBK TO STOP
BRIDGE

ARM TRAVEL LS
2 NORTH OK
2 SOUTH NOT OK

STARTED SEG @ 9:25
250HRS ON CLOCK 4KW GENSET.

ATS WILL NOT TRANSFER
DISCUSSED W/ BOB & PHIL
9:40 LEFT W/ SEG
NOISY BKE LIFTING
OK DOWN.
INTERLOCKS OK
INDICATIONS OK EXCEPT FOR
OIL TEMP